Common Market for Eastern and Southern Africa

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COMESA 292 (2007) (English): Design criteria of overhead transmission lines

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COMESA HARMONISED STANDARD

COMESA/FDHS 292:2007

Design criteria of overhead transmission lines

REFERENCE: FDHS 292:2007

Foreword

The Common Market for Eastern and Southern Africa (COMESA) was established in 1994 as a regional economic grouping consisting of 20 member states after signing the co-operation Treaty. In Chapter 15 of the COMESA Treaty, Member States agreed to co-operate on matters of standardisation and Quality assurance with the aim of facilitating the faster movement of goods and services within the region so as to enhance expansion of intra-COMESA trade and industrial expansion.

Co-operation in standardisation is expected to result into having uniformly harmonised standards. Harmonisation of standards within the region is expected to reduce Technical Barriers to Trade that are normally encountered when goods and services are exchanged between COMESA Member States due to differences in technical requirements. Harmonized COMESA Standards are also expected to result into benefits such as greater industrial productivity and competitiveness, increased agricultural production and food security, a more rational exploitation of natural resources among others.

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COMESA Standards are subject to review, to keep pace with technological advances. Users of the COMESA Harmonized Standards are therefore expected to ensure that they always have the latest version of the standards they are implementing.

This COMESA standard is technically identical to IEC 60826:2003, *Design criteria of overhead transmission lines*

INTERNATIONAL STANDARD

IEC 60826

Third edition
2003-10

Design criteria of overhead transmission lines

*This **English-language** version is derived from the original **bilingual** publication by leaving out all French-language pages. Missing page numbers correspond to the French-language pages.*

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Publication numbering

As from 1 January 1997 all IEC publications are issued with a designation in the 60000 series. For example, IEC 34-1 is now referred to as IEC 60034-1.

Consolidated editions

The IEC is now publishing consolidated versions of its publications. For example, edition numbers 1.0, 1.1 and 1.2 refer, respectively, to the base publication, the base publication incorporating amendment 1 and the base publication incorporating amendments 1 and 2.

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INTERNATIONAL ELECTROTECHNICAL COMMISSION

DESIGN CRITERIA OF OVERHEAD TRANSMISSION LINES

FOREWORD

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International Standard IEC 60826 has been prepared by IEC technical committee 11: Overhead lines.

This third edition cancels and replaces the second edition which was issued as a technical report in 1999. It constitutes a technical revision and now have the status of an International Standard.

This revision consists mainly of splitting the standard into two sections, normative and informative, in addition to simplifying its contents and improving some specific design requirements in accordance with recent technical advances.

The text of this standard is based on the following documents:

FDIS	Report on voting
11/175/FDIS	11/177/RVD

Full information on the voting for the approval of this standard can be found in the report on voting indicated in the above table.

This publication has been drafted in accordance with the ISO/IEC Directives, Part 2.

The committee has decided that the contents of this publication will remain unchanged until 2008. At this date, the publication will be

- reconfirmed;
- withdrawn;
- replaced by a revised edition, or
- amended.

DESIGN CRITERIA OF OVERHEAD TRANSMISSION LINES

1 Scope

This International Standard specifies the loading and strength requirements of overhead lines derived from reliability based design principles. These requirements apply to lines 45 kV and above, but can also be applied to lines with a lower nominal voltage.

This standard also provides a framework for the preparation of national standards dealing with overhead transmission lines, using reliability concepts and employing probabilistic or semi-probabilistic methods. These national standards will need to establish the local climatic data for the use and application of this standard, in addition to other data that are country specific.

Although the design criteria in this standard apply to new lines, many concepts can be used to address the reliability requirements for refurbishment and uprating of existing lines.

This standard does not cover the detailed design of line components such as towers, foundations, conductors or insulators.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

IEC 60652:2002, *Loading tests on overhead line structures*

IEC 61089:1991, *Round wire concentric lay overhead electrical stranded conductors*

IEC 61773:1996, *Overhead lines – Testing of foundations for structures*

IEC 61774:1997, *Overhead lines – Meteorological data for assessing climatic loads*

IEC 61284:1997, *Overhead lines – Requirements and tests for fittings*

3 Terms, definitions, symbols and abbreviations

For the purposes of this document, the following terms, definitions, symbols and abbreviations apply.

3.1 Terms and definitions

3.1.1
characteristic strength
guaranteed strength, minimum strength, minimum failing load
R_c
value guaranteed in appropriate standards

NOTE This value usually corresponds to an exclusion limit, from 2 % to 5 %, with 10 % being an upper practical (and conservative) limit.

3.1.2
coefficient of variation
COV
ratio of the standard deviation to the mean value

NOTE The COV of load and strength are respectively denoted by v_Q and v_R.

3.1.3
components
different parts of a transmission line system having a specified purpose

NOTE Typical components are towers, foundations, conductors and insulator strings.

3.1.4
damage limit (of a component)
serviceability limit state
strength limit of a component corresponding to a defined limit of permanent (or inelastic) deformation of this component which leads to damage to the system if it is exceeded

NOTE This limit is also called the serviceability limit state in building codes based on limit states design.

3.1.5
damage state (of the system)
state where the system needs repairing because one of its components has exceeded its damage limit

NOTE The system needs repairing because it is not capable of fulfilling its task under design loads or because design clearances may be reduced (e.g. conductor to ground).

3.1.6
elements
different parts of a component

NOTE For example, the elements of a steel lattice tower are steel angles, plates and bolts.

3.1.7
exclusion limit
***e* %**
value of a variable taken from its distribution function and corresponding to a probability of *e* % of not being exceeded

3.1.8
failure limit (of a component)
ultimate limit state
strength limit of a component which leads to the failure of the system if this limit is exceeded.

NOTE If this strength limit is exceeded, the system will reach a state called “ultimate limit state” as defined in building codes based on limit states design.

3.1.9
failure state (of the system)
state of a system in which a major component has failed because one of its components has reached its failure limit (such as by rupture, buckling, overturning)

NOTE This state leads to the termination of the ability of the line to transmit power and needs to be repaired.

3.1.10
intact state
state in which a system can accomplish its required function and can sustain limit loads

3.1.11
limit loads
climatic loads corresponding to a return period, T, used for design purposes without additional load factors

NOTE Refer to 5.2.1.

3.1.12
load factor
γ
factor to be multiplied by limit loads in order to design line components

3.1.13
operating period
general measure of useful (or economical) life

NOTE Typical operating periods of transmission lines vary from 30 years to 80 years.

3.1.14
reference wind speed
$\boldsymbol{V_{\mathbf{R}}}$
wind speed at 10 m in height, corresponding to an averaging period of 10 min and having a return period T

NOTE When this wind speed is taken in a terrain type B, which is the most common case in the industry, the reference wind speed is identified as V_{RB}.

3.1.15
reliability (structural)
probability that a system performs a given task, under a set of operating conditions, during a specified time

NOTE Reliability is thus a measure of the success of a system in accomplishing its task. The complement to reliability is the probability of failure or unreliability.

3.1.16
return period (of a climatic event)
average occurrence of a climatic event having a defined intensity

NOTE The inverse of the return period is the yearly frequency which corresponds to the probability of exceeding this climatic event in a given year.

3.1.17
safety
ability of a system not to cause human injuries or loss of lives

NOTE In this standard, safety relates mainly to protection of workers during construction and maintenance operations. The safety of the public and of the environment in general is covered by national regulations.

3.1.18
security (structural)
ability of a system to be protected from a major collapse (cascading effect) if a failure is triggered in a given component

NOTE Security is a deterministic concept as opposed to reliability which is a probabilistic concept.

3.1.19
strength factor
$\boldsymbol{\Phi}$
factor applied to the characteristic strength of a component

NOTE This factor takes into account the coordination of strength, the number of components subjected to maximum load, quality and statistical parameters of components.

3.1.20
system
set of components connected together to form the transmission line

3.1.21
task
function of the system (transmission line), i.e. to transmit power between its two ends

3.1.22
unavailability
inability of a system to accomplish its task

NOTE Unavailability of transmission lines results from structural unreliability as well as from failure due to other events such as landslides, impact of objects, sabotage, defects in material, etc.

3.1.23
use factor
ratio of the actual load (as built) to limit load of a component

NOTE For tangent towers, it is virtually equal to the ratio of actual to maximum design spans (wind or weight) and for angle towers; it also includes the ratio of the sines of the half angles of deviation (actual to design angles).

3.2 Symbols and abbreviations

a	Unit action of wind speed on line elements (Pa or N/m^2)
A_c	Wind force on conductors (N)
A_i	Wind force on insulators (N)
A_t	Wind force acting on a tower panel made of steel angles, A_{tc} for cylindrical tower members (N)
B_i	Reduction factor of the reference wind speed for wind and ice combinations

C_x	Drag coefficient (general form)
C_i	Drag coefficient of ice covered conductors (C_{iL} for low probability and C_{iH} for a high probability)
C_{xc}	Drag coefficient of conductors
C_{xi}	Drag coefficient of insulators
C_{xt}	Drag coefficient of supports C_{xt1}, C_{xt2} for each tower face (C_{xtc} on cylindrical tower members)
COV	Coefficient of variation, also identified as v_x (ratio of standard deviation to mean value)
d	Conductor diameter (m)
d_{tc}	Diameter of cylindrical tower members (m)
D	Equivalent diameter of ice covered conductors (D_H for high probability and D_L for low probability) (m)
e	Exclusion limit (%)
e_N	Exclusion limit of N components in series (%)
$f_{(x)}$	Probability density function of variable x
$F_{(x)}$	Cumulative distribution function of variable x
G	Wind factor (general form)
G_c	Combined wind factor of conductors
G_t	Combined wind factor of towers
G_L	Span factor for wind calculations
g	Unit weight of ice (N/m)
$\overline{g}$	Mean value of yearly maximum ice load (N/m)
g_{max}	Maximum weight of ice per unit length observed during a certain number of years (N/m)
g_R	Reference design ice weight (N/m)
g_H	Ice load having a high probability (N/m)
g_L	Ice load having a low probability (N/m)
h	Height of centre of gravity of a panel in a lattice tower (m)
K_R	Terrain roughness factor
K_d	Factor related to the influence of conductor diameter
K_h	Factor to be multiplied by $\overline{g}$ to account for the influence of height above ground
K_n	Factor to be multiplied by $\overline{g}$ to account for the influence of the number of years with icing observations
l_e	Length of a support member (m)
L	Span length or wind span (m)
L_m	Average span (m)
n	Number of years of observation of a climatic event
N	Number of components subjected to maximum loading intensity
P_f	Probability of failure (%)
P_{fi}	Probability of failure of component i (%)
P_s	Probability of survival (%)
P_{si}	Probability of survival of component i (%)

Q	General expression used to identify the effects of weather related loads on lines and their components
Q_T	The system limit load corresponding a return period T
q_0	Dynamic reference wind pressure due to reference wind speed V_R (q_{0L}, q_{0H} for low and high probability) (Pa or N/m^2)
Re	Reynolds number
R	Strength
$\overline{R}$	Mean strength
R_c	Characteristic strength
$(e)R$	Exclusion limit (e) of strength
RSL	Residual static load
S_i	Projected area of insulators (m^2)
S_t	Projected area of a tower panel (m^2)
t	Ice load expressed in uniform radial ice thickness around the conductor (mm)
t_R	Reference ice load expressed in uniform radial thickness around the conductor (mm)
T	Return period in years
u	Number of standard deviations between mean strength and characteristic strength
U	Use factor
v_x	Coefficient of variation (COV) of variable x
V_m	Yearly maximum wind speed (m/s)
$\overline{V}_m$	Mean yearly maximum wind speed (m/s)
V_G	Yearly maximum gradient wind speed (m/s)
$\overline{V}_G$	Mean yearly maximum gradient wind speed (m/s)
V_R	Reference wind speed (m/s)
V_{iL}	Low probability reference wind speed associated with icing (m/s)
V_{iH}	High probability reference wind speed associated with icing (m/s)
w	Unit weight of conductor or ground wire (N/m)
$\overline{x}$	Mean value of variable x
Y	Horizontal distance between foundations of a support (m)
z	Height above ground of conductors, centre of gravity of towers panels, or insulator strings (m)
γ	Load factor (general form)
γ_U	Use factor coefficient
γ_{TW}	Load factor to adjust the 50 year wind speed to a return period T
γ_{Tit}	Load factor to adjust the 50 year ice thickness to a return period T

$\gamma_{T\mathrm{iw}}$	Load factor to adjust the 50 year ice weight to a return period T
δ	Ice density (kg/m^3)
Φ	Strength factor (general form)
Φ_R	Global strength factor
Φ_N	Strength factor due to number of components subjected to maximum load intensity
Φ_S	Strength factor due to coordination of strength
Φ_Q	Strength factor due to quality
Φ_c	Strength factor related to the characteristic strength R_c
σ_x	Standard deviation of variable x
μ	Mass of air per unit volume (kg/m^3)
τ	Air density correction factor
ν	Kinetic air viscosity (m^2/s)
Ω	Angle between wind direction and the conductor (degrees)
θ	Angle of incidence of wind direction with the tower panel (degrees)
θ'	Angle of incidence of wind direction with cylindrical elements of tower (degrees)
χ	Solidity ratio of a tower panel

4 General

4.1 Objective

This standard serves either of the following purposes:

a) It provides design criteria for overhead lines based on reliability concepts. The reliability based method is particularly useful in areas where significant amounts of meteorological and strength data are readily available. This method may however be used for lines designed to withstand specific climatic loads, either derived from experience or through calibration with existing lines that had a long history of satisfactory performance. In these cases, design consistency between strengths of line components will be achieved, but actual reliability levels may not be known, particularly if there has been no evidence or experience with previous line failures.

   It is important to note that the design criteria in this standard do not constitute a complete design manual for transmission lines. However, guidance is given on how to increase the line reliability if required, and to adjust the strength of individual components to achieve a desired coordination of strength between them.

   This standard also provides minimum safety requirements to protect people from injury, as well as to ensure an acceptable level of service continuity (safe and economical design).

b) It provides a framework for the preparation of national standards for transmission lines using reliability concepts and employing probabilistic or semi-probabilistic methods. These national standards will need to establish the climatic data for the use and application of this standard in addition to other data specific to each country.

The design criteria in this standard apply to new line conditions. It is however a fact of life that transmission lines age and lose strength with time. The amount of strength reduction due to ageing is difficult to generalize, as it varies from one component to another, and also depends on the type of material, the manufacturing processes and the environmental influences. This issue is currently being studied by relevant technical bodies.

The requirements are specified in this standard, while, in Annexes A to C, additional informative data and explanations are given.

4.2 System design

The methodology is based on the concept whereby a transmission line is designed as a system made of components such as supports, foundations, conductors and insulator strings. This approach enables the designer to coordinate the strengths of components within the system and recognizes the fact that a transmission line is a series of components where the failure of any component could lead to the loss of power transmitting capability. It is expected that this approach should lead to an overall economical design without undesirable mismatch.

As a consequence of such a system design approach, it is recognized that line reliability is controlled by that of the least reliable component.

An overhead transmission line can be divided into four major components as shown in Figure 1. Subsequently, each component may be divided into elements.

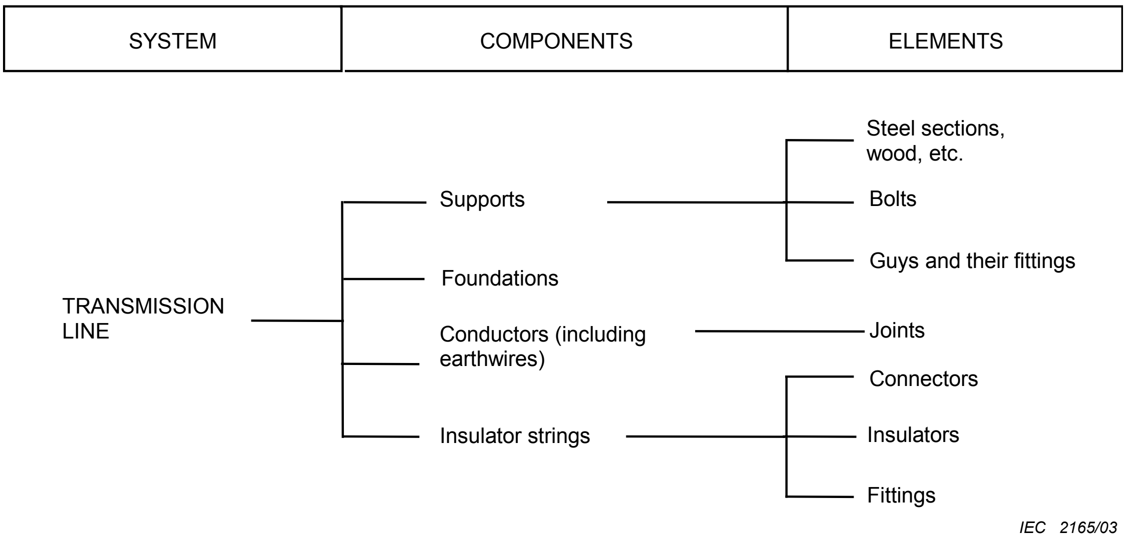


Figure 1 – Diagram of a transmission line

4.3 System reliability

The objective of design criteria described in this standard is to provide for reliable and safe lines. The reliability of lines is achieved by providing strength requirements of line components larger than the quantifiable effects of specified weather related loads. These climatic loads are identified in this standard as well as means to calculate their effects on transmission lines. However, it has to be recognized that other conditions, not dealt with in the design process, can occur and lead to line failure such as impact of objects, defects in material, etc. Some measures, entitled security requirements, included in this standard provide lines with enough strength to reduce damage and its propagation, should it occur.

5 General design criteria

5.1 Methodology

The recommended methodology for designing transmission line components is summarized in Figure 2 and can be described as follows:

a) Collect preliminary line design data and available climatic data.

NOTE 1 In some countries, reference wind speed, such as the 50 year return period, is given in national standards.

b1) Select the reliability level in terms of return period of limit loads.

NOTE 2 Some national regulations and/or codes of practice, sometimes impose directly or indirectly, design requirements that may restrict the choice offered to designers.

b2) Select the security requirements (failure containment).

b3) List safety requirements imposed by mandatory regulations and construction and maintenance loads.

c) Calculate climatic variables corresponding to selected return period of limit loads.

d1) Calculate climatic limit loads on components.

d2) Calculate loads corresponding to security requirements.

d3) Calculate loads related to safety requirements during construction and maintenance.

e) Determine the suitable strength coordination between line components.

f) Select appropriate load and strength factors applicable to load and strength equations.

g) Calculate the characteristic strengths required for components.

h) Design line components for the above strength requirements.

This standard deals with items b) to g). Items a) and h) are not part of the scope of this standard. They are identified by a shaded frame in Figure 2.

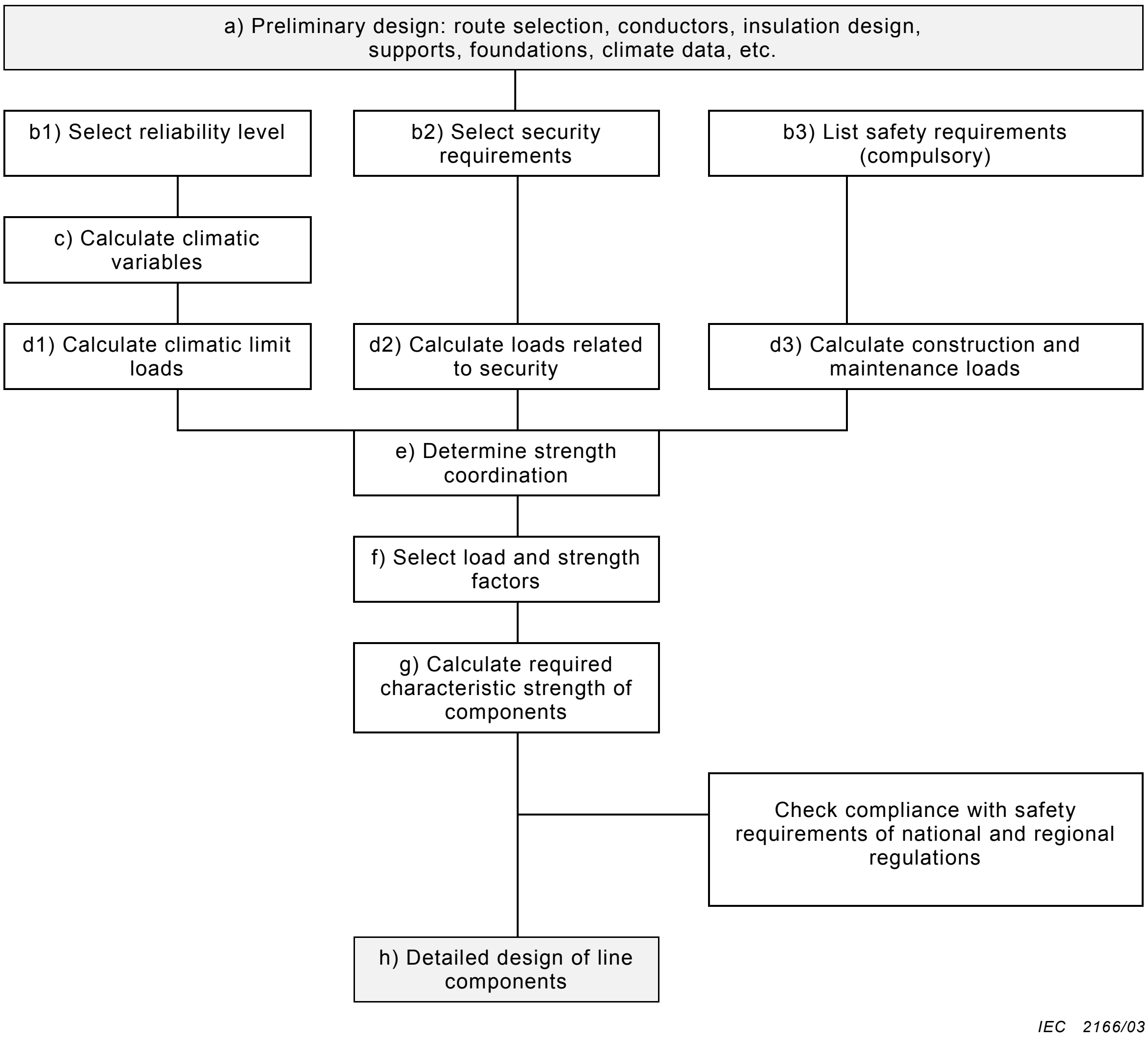


Figure 2 – Transmission line design methodology

5.1.1 Reliability requirements

5.1.1.1 Reliability levels (weather related loads)

Reliability requirements aim to ensure that lines can withstand the defined climatic limit loads (wind, ice, ice and wind, with a return period *T*) and the loads derived from these events during the projected life cycle of the system and can provide service continuity under these conditions.

Transmission lines can be designed for different reliability levels (or classes). For the purposes of this standard, the reference reliability level is defined as the reliability of a line designed for a 50 year return period climatic event associated with a 10 % exclusion limit of strength (applies to the components selected as the least reliable). This reference reliability level is generally regarded as providing an acceptable reliability level in respect of continuity of service and safety.

Lines can be designed for higher reliability levels by increasing the return period *T* of climatic events. A higher reliability can be justified for example by the importance of the line in the network. Three reliability levels are proposed in this standard and are assumed to cover the range of values to be considered for most transmission lines. These levels are expressed in terms of return periods of climatic limit loads as shown in Table 1. For temporary lines, some wooden poles or lines of limited importance, return periods of about 25 years may be appropriate.

Table 1 – Reliability levels for transmission lines

Reliability levels	1	2	3
T, return period of climatic limit loads, in years	50	150	500

NOTE Some national regulations and/or codes of practice, sometimes impose, directly or indirectly, design requirements that may restrict the choice offered to designers.

Other values of *T* in the range of 50-500 years, such as 100, 200 and 400 years, can be used if justified by local conditions.

In some cases, individual utility's requirements can dictate other reliability levels depending on the proper optimization between initial cost of the line and future cost of damage, as well as on uncertainties related to input design parameters.

5.1.1.2 Approximate values for yearly reliability

Both loads (*Q*) and strengths (*R*) are stochastic variables and the combined reliability is computable if the statistical functions of load *Q* and strength *R* are known. The condition for a line to be reliable is when loads effects are less than the strength withstand of the line. The reliability condition translates into the following equation:

Yearly reliability (probability of survival) = 1 - yearly probability of failure =

$$1 - \int_{-0}^{+\infty} f_{(Q)} \times F_{(R)} \times dQ$$

where

$f_{(Q)}$ is the probability density function of load *Q*;

$F_{(R)}$ is the cumulative distribution function of strength *R*.

NOTE The cumulative distribution function is the integral of the probability density function, i.e.

$$F_{(R)} = \int_{-\infty}^{+\infty} f_{(R)} \times dR$$

When the characteristic strength, deemed to be the strength being exceeded with 90 % probability (i.e. the exclusion limit is 10 %), is set equal to the load Q_T (having a return period *T*), various probabilistic combinations lead to a theoretical yearly minimum reliability of around (1 – 1/2*T*). The actual reliability can be different if input data of load and strength are not sufficiently accurate or available (refer to A.1.2 for further discussions on the variation of probability of failure). In the latter case, the absolute reliability may not be known, but its value relative to a reference design may be computed if new line parameters are comparable to the reference values.

5.1.2 Security requirements

Security requirements correspond to special loads and/or measures intended to reduce risk of uncontrollable progressive (or cascading) failures that may extend well beyond an initial failure. These measures are detailed in 6.6.

NOTE Some security measures, such as those providing longitudinal strength of broken conductor loads for failure containment can lead to an increase in reliability to withstand unbalanced ice loads.

5.1.3 Safety requirements

Safety requirements consist of special loads for which line components (mostly support members) have to be designed, to ensure that construction and maintenance operations do not pose additional safety hazards to people. These measures are detailed in 6.5.

5.2 Climatic load-strength requirements

5.2.1 Limit load

Loads associated with climatic events are random variables. Three weather-related loading conditions are recognized: wind, ice, and wind and ice combined. When statistical data of wind and/or ice are available, these can be used to compute the climatic load corresponding to the selected return period T, designated Q_T, for each component exposed to the climatic event under consideration. In the computation process, consideration should be given to the spatial extent of the line, computable directional trends, etc. It is noted that requirements for other events such as earthquakes are not covered in this standard.

Throughout the present standard, the loading Q_T is called the system limit load having a return period T.

In the calculation process for each component, the following condition has to be checked:

$$\text{Design limit load} < \text{design strength} \qquad (1)$$

or, more precisely,

Load factor $\gamma \times$ effect of limit load Q_T **<** strength factor $\phi \times$ characteristic strength R_c.

With the approach proposed, climatic limit loads Q_T are used for design without additional load factors. Consequently, γ is taken equal to 1.

Thus the previous relation becomes:

$$\text{effect of } Q_T < \phi \times R_c \qquad (2)$$

For weather related loads, the effects of Q_T are detailed in 6.2 to 6.4.

Equation (3) is used to compute the minimum value of characteristic strength R_c for each component in order to withstand limit loads.

$$R_c > (\text{effect of } Q_T) / \phi \qquad (3)$$

Q_T can be obtained from the statistical analysis of climatic data in accordance with techniques detailed in Annexes A to C. In some national standards, a reference (usually a 50 year return period value) climatic variable is specified. In such case, the climatic variable for any return period T (years) can be estimated by multiplying the 50 year reference value of the climatic variable by the load factor γ_T, given in Table 2.

Table 2 – Default γ_T factors for adjustment of climatic loads in relation to return period T vs. 50 years

	Wind speed	Ice variable		
Return period *T* years	γ_{TW}	γ_{Tit} **(ice thickness)**	**or**	γ_{Tiw} **(ice weight)**
50	1	1		1
150	1,10	1,15		1,20
500	1,20	1,30		1,45

NOTE The above γ values are sufficiently accurate for a COV of up to 0,16 for wind speed, 0,30 for ice thickness and 0,65 for unit ice weight and are derived from the Gumbel distribution function.

5.2.2 Design requirements for the system

Three types of design conditions shall be checked: reliability, security and safety. Table 3 summarizes the context of loads, the required performance and the limit states associated with each condition.

Table 3 – Design requirements for the system

Condition (or requirement)	Type of load	Required performance	Corresponding limit state
Reliability	Climatic loads due to wind, ice, ice plus wind, with a return period *T*	To ensure reliable and safe power transmission capability	Damage limit
Security	Torsional, vertical, and longitudinal loads	To reduce the probability of uncontrollable propagation of failures (failure containment)	Failure limit
Safety	Construction and maintenance loads	To ensure safe construction and maintenance conditions	Damage limit

5.2.3 Design equation for each component

When designing individual line components, Equation (2) can be expanded into:

$$\gamma_U \times \text{effect of } Q_T < \phi_R \times R_c \qquad (4)$$

γ_U is the use factor coefficient. It is derived from the distribution function of the use factor U and expresses the relationship between effective (actual) and design (original) conditions or parameters. The use factor U is a random variable equal to the ratio of the effective (actual line conditions) limit load applied to a component by a climatic event to the design limit load

for this component under the same climatic event (using maximum parameters). Symbol γ_U is introduced because components are designed by families, not individually. However, since components are usually designed prior to specific knowledge of their real line parameters, it is admissible to use $\gamma_U = 1$ for new lines design.

NOTE This is equivalent to considering that design is influenced by the maximum span in the line.

It is important to note this simplification will certainly have a positive influence on reliability. However, the influence of γ_U on reliability can be fully considered for existing lines, where use parameters of components are fully known. For a detailed discussion on the subject, refer to Clause B.4.

R_c is the characteristic strength. It is the value guaranteed in appropriate standards for new components, usually with a 90 % to 98 % probability. This value is also called the guaranteed strength, the minimum strength or the minimum failing load. When not specified or calculated, the exclusion limit of R_c can be conservatively taken as 10 % (typical values are in the range of 2 % to 10 %). It is generally accepted that line components will age with time, just like any structural components, and will suffer a reduction in their strength. This quantification of loss of strength with time is not covered in this standard and the reliability values suggested herein are based on new line conditions.

Very often, standards only provide a single normative value usually associated with failure of the component, while the approach mentioned above requires the consideration of two limits: damage and failure limits. If the damage limit corresponding to R_c is not specified in the standards, Tables 14 to 17 can be used to provide such values.

ϕ_R is a global strength factor applicable to the component being designed that takes into account:

a) Features related to the system
   - the number (N) of components exposed to the limit load Q_T during any single occurrence of this load event, (hence ϕ_N);
   - the coordination of strengths selected between components, (hence ϕ_S).

b) Features related to the component
   - the difference in the quality of the component during prototype testing and actual installation, (hence ϕ_Q);
   - the difference between the actual exclusion limit of R_c and the supposed e = 10 %, (hence ϕ_c);

As these factors are statistically independent:

$$\phi_R = \phi_N \times \phi_S \times \phi_Q \times \phi_c \tag{5}$$

All the above strength factors ϕ are detailed in 7.2.

6 Loadings

6.1 Description

This clause defines structural loadings considered for the design of transmission line components.

As indicated in 5.2.1, three load categories are considered:

a) Loads due to climatic events or any loads derived from them which govern the reliability of the line for the expected life time.

   These loads will be analysed in the following subclauses:

   - wind loads (6.2);
   - ice without wind (6.3);
   - ice with wind (6.4).

b) Loads related to safety requirements (construction and maintenance) (6.5);

c) Loads related to security requirements (failure containment) (6.6).

6.2 Climatic loads, wind and associated temperatures

This subclause defines the procedures to evaluate the wind and associated temperature effects on line components and elements (conductors, insulator strings, supports).

6.2.1 Field of application

Although this subclause applies in principle to any overhead line, it is most accurately defined for the following conditions:

- Span lengths between 200 m and 800 m. Calculations of the various coefficients (in particular for gusty winds) have to be checked for span lengths outside this range. However, for span lengths greater than 800 m, a gust coefficient corresponding to 800 m span could be safely chosen. For span lengths less than 200 m, the values applicable to 200 m span can be applied.
- Height of supports less than 60 m. Taller supports could be designed following the same principles, but the calculated wind actions would need to be checked. In particular, the eigen frequency of structures above 60 m will often increase the gust response factor.
- Altitude of crossed areas not exceeding 1 300 m above the average level of the topographic environment, except where specific study results are available
- Terrain without local topographical features whose size and shape are likely to significantly affect the wind profile of the region under consideration.

It is important to note that requirements for winds associated with localized events such as tornadoes are not specifically covered in this standard. These winds can cause serious damage to transmission lines either directly (due to wind forces) or indirectly (due to impact of wind carried objects). Furthermore, the effects of acceleration due to funnelling between hills or due to sloping grounds are not covered and may require specific studies to assess such influences.

6.2.2 Terrain roughness

Wind speed and turbulence depends on the terrain roughness. With increasing terrain roughness, turbulence increases and wind speed decreases near ground level. Four types of terrain categories, with increasing roughness values, are considered in this standard as indicated in Table 4.

Table 4 – Classification of terrain categories

Terrain category	Roughness characteristics	K_R
A	Large stretch of water upwind, flat coastal areas	1,08
B	Open country with very few obstacles, for example airports or cultivated fields with few trees or buildings	1,00
C	Terrain with numerous small obstacles of low height (hedges, trees and buildings)	0,85
D	Suburban areas or terrain with many tall trees	0,67

In Table 4, the roughness factor K_R represents a multiplier of the reference wind speed for conversion from one terrain category to another. The use of K_R is detailed in 6.2.3. A description of terrain roughness characteristics is given in Clause A.4.

6.2.3 Reference wind speed V_R

V_R is defined as the reference wind speed (m/s) corresponding to a return period T. V_R can be determined from a statistical analysis of relevant wind speed data at 10 m above ground and with an averaging period of 10 min. Usually V_R is measured in weather stations typical of terrain type B, such as airports. In such cases, V_R is identified as V_{RB}. Where available wind data differs from these assumptions, refer to Clause A.4 for conversion methods. If the reference wind speed for terrain category B, V_{RB} is only known, V_R can be determined with $V_R = K_R\ V_{RB}$, where K_R is the roughness factor.

NOTE Some countries, such as the USA, have recently switched to the 3 s averaging period. The conversion from the 3 s to the 10 min wind can be derived from available wind statistics, or if lacking, from Figure A.7

6.2.4 Combination of wind speed and temperatures

Unless a strong positive correlation is established between wind speed and temperature, it is assumed that maximum wind speed does not usually occur with minimum temperature. Consequently, only two combinations shall normally be considered for design purposes; the first being maximum wind at average daily minimum temperature and the second being reduced wind at the extreme minimum temperature.

In practice, the following two combinations need to be checked:

a) High wind speed plus reference temperature condition

The wind velocities defined above for computation shall be considered as occurring at an air temperature equal to the average of the daily minimum temperatures, peculiar to the site.

b) Reduced wind speed at the minimum low temperature condition

b1) Reduced wind speed

The reduced wind speed is equal to the reference wind speed V_R multiplied by a coefficient chosen according to local meteorological conditions. When there is no reliable knowledge of local conditions a value of 0,6 for this coefficient is suggested.

b2) Temperature associated with the reduced wind speed

The minimum temperature shall be considered as being equal to the minimum yearly value, having a return period of T years.

It is noted that the design of transmission lines is not generally controlled by the combination of reduced wind speed and minimum low temperatures. This loading case may therefore be omitted, except for cases of supports with very short spans (typically less than 200 m) and minimum low temperatures (typically below –30 °C), or dead-end supports.

6.2.5 Unit action of the wind speed on any line component or element

The characteristic value a of the unit action in Pa (N/m^2), due to the wind blowing at right angles to any line component or element (conductors, insulator strings, all or part of the support) is given by the following expression:

$$a = q_0 \, C_x \, G \tag{6}$$

q_0 is the dynamic reference wind pressure (in Pa or N/m^2) and is given in terms of the reference wind speed V_R modified by roughness factor K_R (see Table 4) corresponding to the terrain category at the location of the line:

$$q_0 = \frac{1}{2}\tau\mu\left(K_R V_{RB}\right)^2 \quad (V_{RB} \text{ in m/s, and } q_0 \text{ in N/m}^2) \tag{7}$$

μ is the air mass per unit volume equal to 1,225 kg/m^3 at a temperature of 15 °C and an atmospheric pressure of 101,3 kPa at sea level;

τ is the air density correction factor. When limit wind speeds are known to be strongly correlated with an altitude and/or temperature significantly different from the assumptions of 15 °C and sea level, the correction factor τ given in Table 5 can be applied to the pressure q_0, otherwise, τ is considered to be equal to 1;

C_x is the drag (or pressure) coefficient depending on the shape and surface properties of the element being considered;

G is the combined wind factor, taking into account the influences of the height of the element above ground level, terrain category, wind gusts and dynamic response (component effect). In the case of conductor loads, this factor shall be split into two factors G_L and G_c.

These factors shall be considered separately for each line component or element.

Table 5 – Correction factor τ of dynamic reference wind pressure q_0 due to altitude and temperatures

Temperature °C	**Altitude** m			
	0	**1 000**	**2 000**	**3 000**
30	0,95	0,84	0,75	0,66
15	1,00	0,89	0,79	0,69
0	1,04	0,94	0,83	0,73
–15	1,12	0,99	0,88	0,77
–30	1,19	1,05	0,93	0,82
NOTE The reference value corresponds to 0 m altitude and a temperature of 15 °C.				

6.2.6 Evaluation of wind loads on line components and elements

6.2.6.1 Wind loads on conductors

Wind effects on conductors consist of loads due to wind pressure as well as the effect of the increase in the mechanical tension.

The load (A_c) in N due to the effect of the wind pressure upon a wind span L, applied at the support and blowing at an angle Ω with the conductors, is given by the following expression, using q_0 of Equation (7).

$$A_c = q_0 \, C_{xc} \, G_c \, G_L \, d \, L \sin^2 \Omega \qquad (8)$$

where

- C_{xc} is the drag coefficient of the conductor taken equal to 1,00 for the generally considered stranded conductors and wind velocities. Other values can be used if derived from direct measurements or wind tunnel tests.
- G_c is the combined wind factor for the conductors given in Figure 3, which depends on height z and terrain categories.
- G_L is the span factor given in Figure 4.
- d is the diameter of the conductor (m).
- L is the wind span of the support, equal to half the sum of the length of adjacent spans of the support.
- Ω is the angle between the wind direction and the conductor (Figure 6).

The total effect of the wind upon bundle conductors shall be taken as equal to the sum of the actions on the sub-conductors without accounting for a possible masking effect of one of the sub-conductors on another.

The height to be considered for conductors is the center of gravity of the suspended conductor theoretically located at the lower third of the sag. For the purpose of transmission support calculations, it is acceptable to consider z equal to the height of attachment point of the conductor at the support (for flat configuration) or of the middle conductor (for double circuit configuration). These assumptions for conductors are conservative and compensate for the increased height of the ground wire on top of the support.

The general equation for computing conductor wind forces on an angle support is given in A.4.6.

Values different from Figures 3 and 4 can be used if supported by data and validated models.

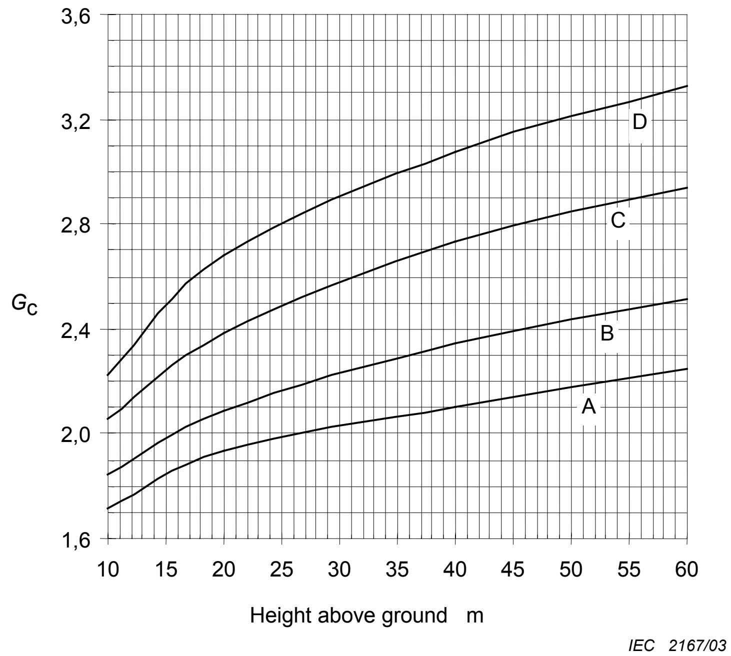


Figure 3 – Combined wind factor G_c for conductors for various terrain categories and heights above ground

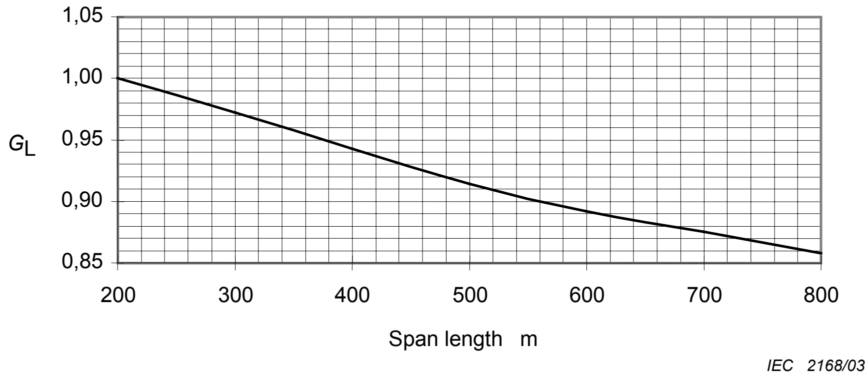


Figure 4 – Span factor G_L

6.2.6.2 Wind effect on conductor tension

Wind acting on conductors will cause an increase in their mechanical tension that can be computed with standard sag-tension methods. Two cases of wind and temperature combinations shall be checked, as stated in 6.2.4.

If a series of spans is separated by suspension insulators, the ruling span concept may be used for tension calculations. It is important to note that the ruling span concept implies that the same wind pressure applies to all spans between dead-end insulators. This assumption becomes more conservative with an increasing number of suspension spans and length of insulator strings. In some cases the wind load calculated with Equation 8 can be reduced, if supported by experience or data, but in no case by more than 40 %. With regard to ground wires, no reduction of wind pressure is applicable because the absence of suspension insulator strings prevents equilibrium of horizontal tensions even at suspension supports, hence, results in the inapplicability of the ruling span concept.

NOTE 1 The ruling span of a series of suspension spans between dead-ends, is equal to $(\Sigma L^3/\Sigma L)^{1/2}$.

NOTE 2 Caution should be exerted when using the above reduction factor of up to 40 % because some supports may be used in sections with few suspension spans and even as a single span between dead-end towers, in such case, no reduction factor is applied.

6.2.6.3 Wind loads on insulators strings

Wind loads acting on insulator strings originate from the load A_c transferred by the conductors and from the wind pressure acting directly on the insulator strings. The latter load is applied conventionally at the attachment point to the support in the direction of the wind and its value (in N) is given by:

$$A_i = q_0\ C_{xi}\ G_t\ S_i \tag{9}$$

where

q_0 is the dynamic reference wind pressure in Pa (N/m^2);

C_{xi} is the drag coefficient of the insulators, considered equal to 1,20;

G_t is the combined wind factor given in Figure 5, variable with the roughness of the terrain, and with the height of the centre of gravity of the insulator string above the surrounding land. The same average height of conductors can be used.

S_i is the area of the insulator string projected horizontally on a vertical plane parallel to the axis of the string (m^2). In the case of multiple strings, the total area can be conservatively taken as the sum of all strings.

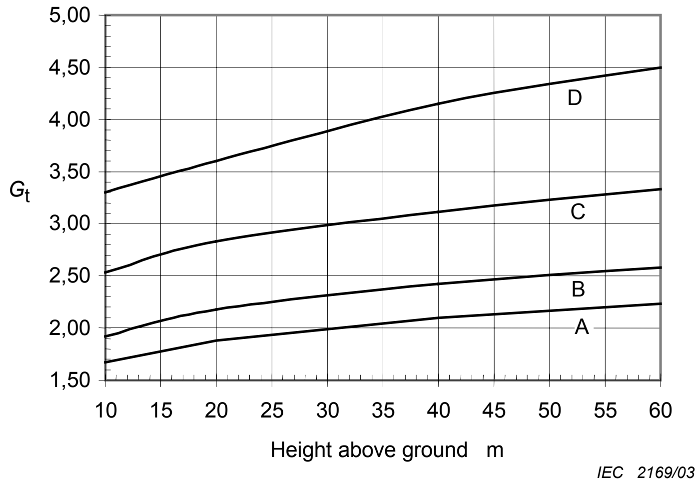


Figure 5 – combined wind factor G_t applicable to supports and insulator strings

It is noted that wind on insulator strings has a small effect on design of supports. Consequently, it may be acceptable for most lines to simplify the calculation of wind pressure by conservatively adopting the same pressure as the one applied to supports.

6.2.6.4 Wind loads on supports

Wind loads on the supports consist of the wind loads transmitted by conductors and insulators as well as the wind loads acting on the support itself.

The method of determination of wind loadings on the support itself is only given in this standard for the most common types of supports, i.e. lattice towers and towers with cylindrical elements. This method can, however, be applied to other types of supports.

During detailed design of supports, an iterative process is required in order to compute wind loads on supports. This is due to the fact that the projected area of members is only known after completion of the detailed design.

6.2.6.4.1 Lattice towers of rectangular cross-section

In order to determine the effect of the wind on the lattice tower itself, the latter is divided into different panels. Panel heights are normally taken between the intersections of the legs and bracing.

For a lattice tower of square/rectangular cross-section, the wind loading A_t (in N), in the direction of the wind, applied at the centre of gravity of this panel, made up of various support members, is equal to:

$$A_t = q_0\ (1 + 0{,}2 \sin^2 2\theta)\ (S_{t1}\ C_{xt1} \cos^2 \theta) + S_{t2}\ \mathrm{C}_{xt2} \sin^2 \theta)\ G_t \qquad (10)$$

where

q_0	is the dynamic reference wind pressure Pa (N/m^2), see Equation (7);
θ	is the angle of incidence of the wind direction with the perpendicular to face 1 of the panel in a horizontal plane (Figure 6);
S_{t1}	is the total surface area projected normally on face 1 of the panel (m^2)
S_{t2}	is the total surface area projected normally on face 2 of the support members of face 2 of the same panel (m^2). The projections of the bracing elements of the adjacent faces and of the diaphragm bracing members can be neglected when determining the projected surface area of a face.
C_{xt1} , C_{xt2}	are the drag coefficients peculiar to faces 1 and 2 for a wind perpendicular to each face. C_{xt1} , C_{xt2} are given in Figure 7 for panels of the tower where all or some of the members exposed have plane surfaces, and in Figure 8 where all support members have a circular section.
χ	is the solidity ratio of a panel equal to the projected area of members divided by the total panel area. The solidity ratio χ of one face is the ratio between the total surface of the support members (S_{t1} or S_{t2}), defined above, and the circumscribed area of the face of the considered panel.
G_t	is the combined wind factor for the supports given in Figure 5. The height above ground is measures at the centre of gravity of the panel.

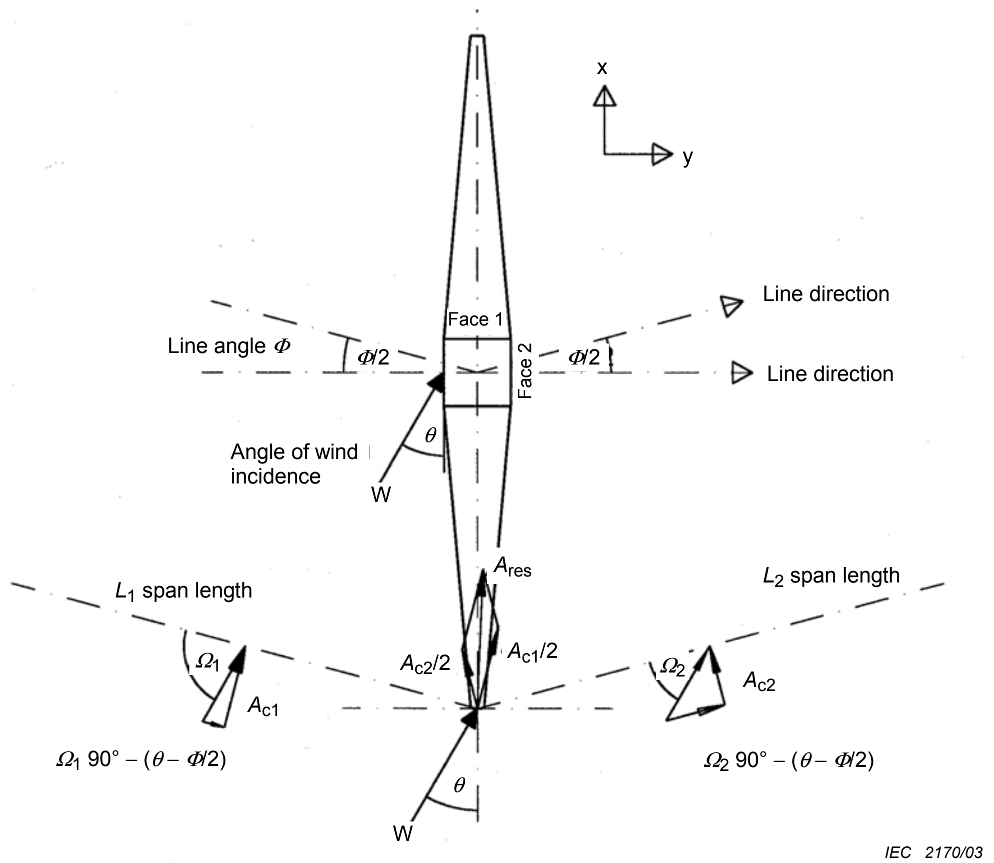


Figure 6 – Definition of the angle of incidence of wind

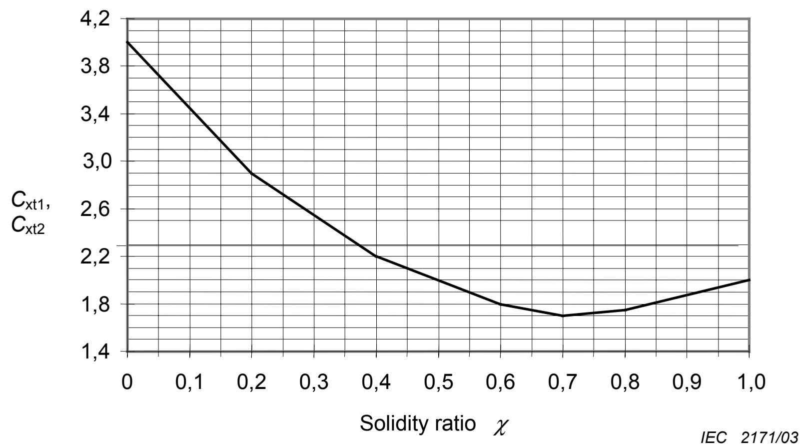


Figure 7 – Drag coefficient C_{xt} for lattice supports made of flat sided members

6.2.6.4.2 Supports with cylindrical members having a large diameter (d_t > 0,2 m)

For such supports the effect of the wind loading (in N) in the direction of the wind, on each member l_e long, applied at the centre of gravity of the member, is equal to:

$$A_{tc} = q_0\ C_{xt}\ G_t\ d_t\ l_e \sin^3 \theta' \quad (11)$$

where

θ' is the angle formed by the direction of the wind and the cylinder axis;

d_t is the diameter of the cylinder (m);

l_e is the length of the member (m);

G_t is the combined wind factor, a function of the terrain category and the height h of the centre of gravity of the member above the ground (Figure 5);

C_{xtc} is the drag coefficient for a wind perpendicular to the axis of the cylinder. The value of C_{xtc} depends on the Reynolds number Re corresponding to the gust speed at this height, and on the roughness of the cylinder. An acceptable simplification is to consider the most unfavourable case of a rough cylinder. The value of C_{xtc} is given in Figure 9 in terms of Re that corresponds to the reference wind speed V_R at this height h (corrections with height are described in A.4) and is given by:

$$Re = \frac{d_t \times V_R}{\nu} \quad (12)$$

ν is the kinetic air viscosity (ν = 1,45 × 10^{-5} m²/s at 15 °C)

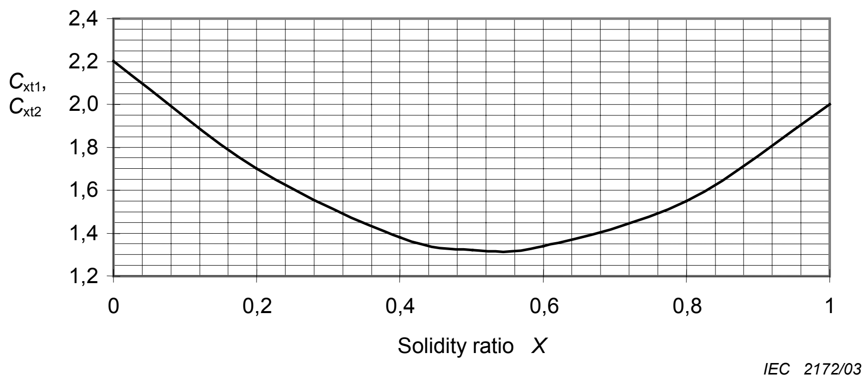


Figure 8 – Drag coefficient C_{xt} for lattice supports made of rounded members

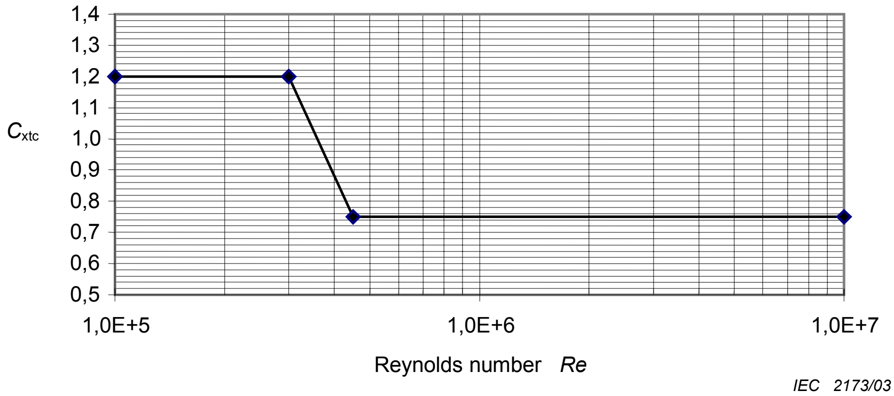


Figure 9 – Drag coefficient C_{xtc} of cylindrical elements having a large diameter

6.3 Climatic loads, ice without wind

6.3.1 Description

Ice loads consist of all combinations of frozen water that adheres to transmission lines such as freezing rain, in-cloud-icing, wet snow, etc. (see description in Clause A.5). This standard covers two main types of icing: precipitation icing and in-cloud icing.

In mountains or regions where both types of icing may occur, the different data for the two types may be treated separately, with separate distributions to provide the basis for the design load. If a difference between the design loads for the two types of ice is apparent, the less important may be ignored, and the more important may take care of combined occurrences.

Although significant loadings due to the presence of ice also involve some wind during and after an icing event, ice only is first considered here, to establish reference conditions that will serve as a basis for the wind and ice combined loadings given in 6.4 as well as non-uniform ice conditions described in 6.3.6.3.

6.3.2 Ice data

Ice load is a random variable that is usually expressed either as a weight per unit length of conductor, *g* (N/m), or as a uniform radial thickness *t* (mm) around conductors and ground wires. In real conditions, ice accretion is random in both shape and density and depends on the type of accretion as indicated in Table A.10 in A.5.4. However, for ease of calculations, these are converted to an equivalent radial ice thickness (*t*) around conductors with a relative density δ of 0,9. Equation (13) expresses the relation between *g* and *t*:

$$g = 9{,}82 \times 10^{-3}\, \delta\, \pi\, t\, (d + t/1\,000) \tag{13}$$

where

g is the ice weight per unit length (N/m);

δ is the ice density (kg/m^3);

t radial ice thickness, assumed uniform around the conductor (mm);

d is the conductor diameter (m).

For an ice density δ = 900 kg/m^3, Equation (13) becomes:

$$g = 27{,}7\ t\ (d + t/1\ 000)$$

When both t and d are expressed in mm and δ = 900 kg/m^3, Equation (13) becomes:

$$g = 0{,}0277\ t\ (t + d)$$

with g in N/m.

Ice load should ideally be deduced from measurements taken from conductors and locations representative of the line. These measurement techniques are described in IEC 61774. Ice accretion models can also supplement direct ice data measurement, but require appropriate validation with real data.

A very important factor with ice accretion is the effect of the terrain. It is difficult to transfer knowledge acquired from one site to another because the terrain strongly influences the icing mechanism.

For design purposes, icing data from measuring stations near or identical to the line site are ideally required. Very often, this will not be the case and service experience with existing installations will provide additional input.

Ice accretion on structures should be considered (refer to A.5.8.2 for suggested method).

NOTE It is noted that weight of ice on lattice steel structures can be quite significant and can reach or exceed the weight of the structure itself in case of radial ice thickness greater than 30-40 mm.

6.3.3 Evaluation of yearly maximum ice load by means of meteorological data analysis

Sufficient data for using the statistical approach in this standard may be obtained by means of an analysis of available standard weather or climatological data over a period of 20 years or more, combined with at least five years of ice observation on the transmission line sites.

If a reliable ice accretion model is available to estimate values for yearly maximum ice loads during a certain number of years, this model can be used to generate ice data which will be used in the statistical analysis. Information about the line site which is necessary to validate and adjust the predicting model may be taken from past experience with existing transmission or distribution lines, from field observations or from the effect of icing on vegetation.

Such a predicting model can be rather simple or become sophisticated, depending on icing severity, terrain, local weather, number or types of ice data collecting sites.

6.3.4 Reference limit ice load

6.3.4.1 Based on statistical data

The reference design load g_R, or t_R if ice thickness is chosen as the ice variable, are the reference limit ice loads corresponding to the selected return period T (function of the reliability level of the line). The g_R or t_R values can be directly obtained from the statistical analysis of data obtained either from direct measurements, icing models, or appropriate combinations of both.

NOTE 1 The figures and equations given in this subclause are based on g_R (N/m) being the ice variable. However, Equation (13) can be used to convert from g_R to t_R if the latter is chosen as the ice variable.

If data is measured (or model simulated) on conductor diameters and heights typical of the line, there will not be any further adjustment to this value. However, if data is measured at the assumed reference height of 10 m on a 30 mm conductor diameter, g_R should be adjusted by multiplying it with a diameter factor K_d and a height factor K_h applicable to the actual line conditions.

K_d is given in Figure 10.

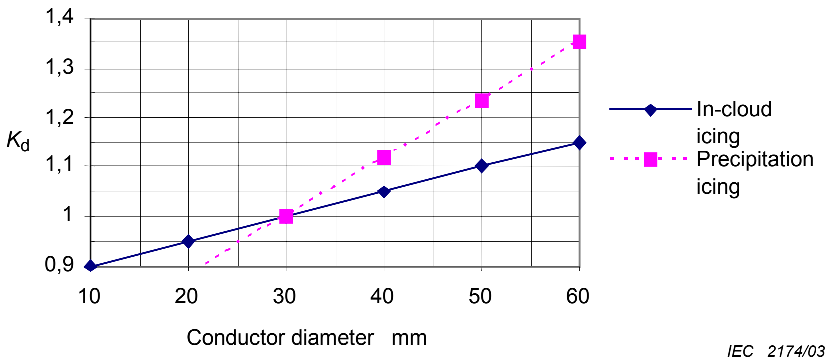


Figure 10 – Factor K_d related to the conductor diameter

For both types of icing, when $K_d \times \bar{g}$ exceeds 100 N/m, the value of K_d is no longer increased. If $\bar{g}$ (average of yearly maximum values of g) is above 100 N/m and d greater than 30 mm, K_d is considered equal 1,0.

K_h describes the variation of g with the height of conductors above the ground. Its value is given in Figure 11.

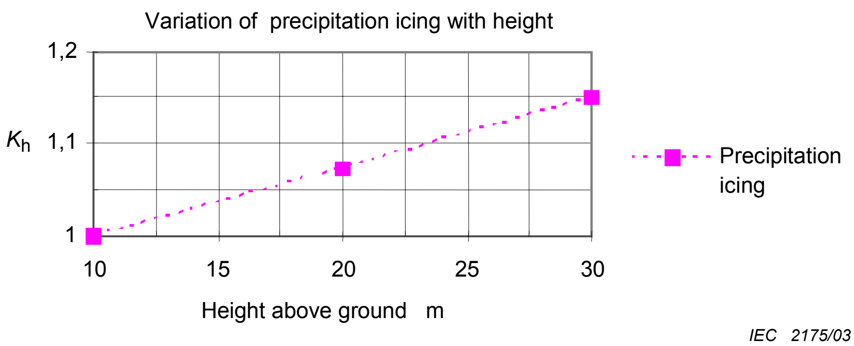


Figure 11 – Factor K_h related to the conductor height

As a simplification, it is suggested that the value g_R be the same for phase conductors and ground wires in the same span, but there is growing evidence that the higher ground wire may accumulate more ice for some types of ice accretion. For variation of in-cloud icing accretion with height, refer to comments in A.5.8.2.

NOTE 2 Some recent studies suggest that bundled conductors may collect less wet snow or in-cloud ice than single conductors due to the difference in torsional behaviour. This matter is currently under investigation.

6.3.4.2 Based on service experience

Where icing data or reliable ice accretion models are not available, the only alternative is to rely on service experienced based on actual ice loads observed on the conductors or deduced from failure events. In both cases, neither the return period of the ice loads, nor the level of reliability will be known.

6.3.5 Temperature during icing

The temperature to be considered with ice conditions shall be –5 °C.

6.3.6 Loads on support

Three different icing conditions on the conductors shall be considered when determining the loads on the support. These are considered to be the most significant and encompass the majority of the icing conditions that are likely to occur:

– uniform ice formation: weight condition;
– non-uniform ice formation: longitudinal and transverse bending condition;
– non-uniform ice formation: torsion condition.

6.3.6.1 Loading cases description

In the description of the different loading conditions, the value of the ice loads are given as functions of the reference design ice load g_R. It is important to be aware of the fact that g_R may vary from one span to another in a section of a line, due to local terrain effects, giving non-uniform situations. The aim is to propose conventional loading conditions for the purpose of calculating conductor tensions which are typical for known occurrences of ice loading.

When computing loads on a support from conductors, the effects of the swing of the insulator set, deflection or rotation of the support and/or foundations and the interaction with other conductors shall be considered.

Ice may not accumulate or shed uniformly from adjacent spans. A non-uniform ice formation is defined as an ice load corresponding to the probability of an ice accretion on up to three spans on one side of the support, whilst on the other spans in the clause the ice is reduced to a certain percentage of that value.

NOTE Unbalanced ice loads due to unequal accretion or ice shedding will invariably occur during icing events. Statistics of unbalanced ice loads are not usually available; however, the recommendations given in this standard should be sufficient to simulate typical unbalanced ice loads that occur such conditions.

6.3.6.2 Uniform ice formation – Maximum weight condition

The maximum uniform ice loading on the conductors is assumed to occur, when the conductor ice loading is equivalent to the reference limit ice load (g_R). The overload per unit length is g_R (N/m), and the total conductor load per unit length = $w + g_R$ (w is the unit weight of conductors in N/m).

6.3.6.3 Non-uniform ice formation on phase conductors and ground wires

Unequal ice accumulations or shedding in adjacent spans will induce critical out-of-balance longitudinal loads on the supports. Unbalanced ice loads can occur either during ice accretion, e.g. in-cloud icing with significant changes in elevation or exposure, or during ice shedding.

Suggested configurations of non-uniform icing conditions are described in Table 6 for support types shown in Figure 12.

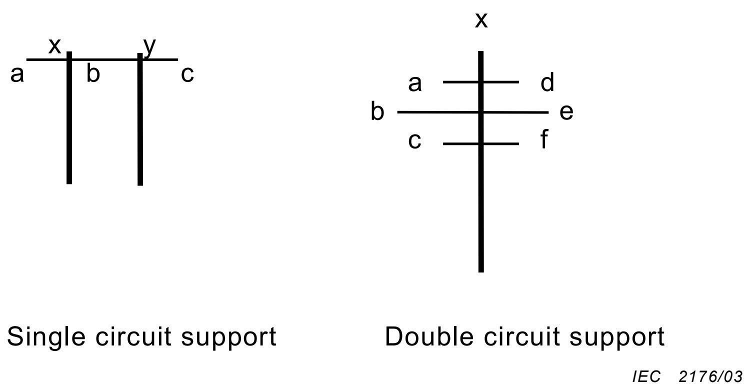


Figure 12 – Typical support types

NOTE For multi-circuit lines, the number of phases subject to non-uniform ice can be different, but not less than that given for double circuit lines.

Table 6 – Non-uniform ice loading conditions

Type of supports	Longitudinal bending condition		Transverse bending condition		Torsional condition	
	Left span	**Right span**	**Left span**	**Right span**	**Left span**	**Right span**
Single circuit	xyabc	XYABC	XYabC	xYabC	XYabC	XYABC
Double circuit	xabcdef	XABCDEF	XabcDEF	XabcDEF	XabcDEF	XABCDEF

NOTE In this table, the letters A, B, C, D, E, F, X, Y represent conductors and spans loaded with 0,7 g_R, while the letters a, b, c, d, e, f, x, y represent conductors and spans loaded with 0,4 × 0,7 g_R . Factors 0,7 and 0,4 are suggested and other values can be used as substantiated by experience.

Where the exposure of the line to its surroundings changes from one span to another, unbalanced loads larger than those described above should be considered. During calculations of longitudinal loads on structures due to unbalanced ice loads, the flexibility of structures and insulator movement shall be taken into account to calculate the resulting longitudinal forces. Use of simplified assumptions is accepted as long as they lead to conservative results.

Where specific sections of an OHL are exposed to severe in-cloud icing and adjacent spans have different levels of moisture-laden winds, it may be applicable to consider maximum ice loading on one side of the support and bare conductors on the other side.

6.4 Climatic loads, combined wind and ice loadings

The combined wind and ice loadings treated in this subclause relate to wind on ice-covered conductors. Wind on ice-covered supports and insulator strings can, if necessary, be treated in a similar way with special attention to drag coefficients.

6.4.1 Combined probabilities – Principle proposed

The action of wind on ice-covered conductors involves at least three variables: wind speed that occurs in presence with icing, ice weight and ice shape (effect of drag coefficient). This action results in both transversal and vertical loads.

Ideally, statistics of wind speed during ice presence on conductors should be used to generate the combined loadings of ice and wind corresponding to the selected reliability level. Since detailed data and observations on ice weight, ice shape and coincident wind, are not commonly available, it is proposed to combine these variables in such a way that the resulting load combinations will have the same return periods T as those adopted for each reliability level.

Assuming that maximum loads are most likely to be related to combinations involving at least one maximum value of a variable (either of wind speed, ice weight or ice shape), a simplified method is proposed: a low probability (index L) value of a variable is combined with high probability (index H) values of the other two variables, as is shown in Table 7. This simplification is equivalent to associating one variable (e.g. ice load) having a return period T with the average of yearly values of all the other variables related to this loading case, such as wind during icing or the drag coefficient.

Table 7 – Return period of combined ice and wind load

Reliability level	**Return period *T*** years	**Return period of the variable having a low probability of occurrence** (index L)	**Return period of remaining variables** (index H)
1	50	50	Average of yearly maximum values
2	150	150	Average of yearly maximum values
3	500	500	Average of yearly maximum values

The density of ice varies with the type of icing and it is recommended that low density ice be combined with the high probability drag coefficient and vice-versa.

Usually, the combination of a low probability drag coefficient (highest value of C_i , or C_{iL}) with a high probability ice and a high probability wind does not constitute a critical loading case. However, if previous service experience or calculation confirms that this load combination can be critical, it should be considered for design purposes.

Consequently two loading combinations will be considered in this standard: Low ice probability (return period T) associated with the average of yearly maximum winds during icing presence, and low probability wind during icing (return period T) associated with the average of the maximum yearly icing.

The low probability (reference values) of ice or wind have already been dealt with separately in the previous paragraphs. These should correspond to the return period T selected for design purposes.

With regard to wind, it is important to note that wind data to be considered is when icing is present on conductors. Such data is not usually available and it is generally accepted to deduce it from the yearly wind statistics.

6.4.2 Determination of ice load

The two main types, precipitation and in-cloud icing, require a separate determination of the maximum ice load associated with wind.

If there is almost no data on combined wind and ice, it can be assumed that $g_L = g_R$ and $g_H = 0{,}40\ g_R$. If combined wind and ice data are available, statistical methods can be used to estimate values for combined variables corresponding to the selected return period T or to the average of yearly maximum winds.

6.4.3 Determination of coincident temperature

The temperature to be considered for combined wind and ice conditions shall be –5 °C for all types of icing.

6.4.4 Determination of wind speed associated with icing conditions

6.4.4.1 Freezing rain (precipitation icing)

Wind velocities associated with icing episodes can be calculated from data, if available or, when there is little or no data, from the following assumptions. In the latter case, the reference wind speed is multiplied by a reduction factor B_i as indicated below:

$Vi_L = B_i \times V_R$, where B_i = (0,60 to 0,85).. This range of B_i is assumed to correspond to the reference wind speed (T= 50, 150 or 500 years) during icing persistence on conductors.

$Vi_H = B_i \times V_R$, where B_i.= (0,4 to 0,5). This range of B_i is assumed to correspond to the average of yearly maximum wind speed during icing persistence on the conductors.

The given range of values in the above equations represents typical values of wind speed during icing periods and takes into account the relative rarity of maximum wind speed during icing periods.

When combined data are available, the process described for wind or ice loading can be used to select a value corresponding to a return period T for each of the expected types of icing.

When wind speed data is not strictly correlated with icing, one should determine the associated maximum wind speed by using the yearly maximum wind speed recorded during freezing precipitation and the following period whilst the air temperature remains below 0 °C (suggested maximum period 72 h).

6.4.4.2 Wet snow (precipitation icing)

Based on both local meteorological conditions and experience, the reduction in the wind speed (V_R) can be determined in a similar manner to that described for freezing rain (see 6.4.4.1). In the absence of specific experience or data, it is suggested to use the same reduction factors as for freezing rain.

6.4.4.3 Dry snow (precipitation icing)

In the absence of any specific data for dry snow, the same values stated for wet snow may be used.

6.4.4.4 Hard rime (in-cloud icing)

In certain areas, hill tops, for example, the maximum rime ice accretion on the conductors usually occurs with the maximum wind speed associated with in-cloud icing. However, in other areas the maximum ice accretion usually occurs under relative load wind speeds.

Basic meteorological and terrain information should be used to evaluate the probability of severe in-cloud icing along the line route, and the corresponding data should be introduced in the calculations. Otherwise, the values given for freezing rain may be used.

6.4.5 Drag coefficients of ice-covered conductors

Wherever possible, drag coefficients for ice covered conductors should be based on actual measured values. In the absence of this data, the effective drag coefficients and ice densities are given in Table 8.

Table 8 – Drag coefficients of ice-covered conductors

	(Precipitation) Wet snow	(In-cloud) Soft rime	(In-cloud) Hard rime	(Precipitation) Glaze ice
Effective drag coefficient C_{iH}	1,0	1,2	1,1	1,0
Associated ice density δ (kg/m^3)	600	600	900	900

The effective drag coefficient is a multiplying factor on the assumed cylindrical shape for the specified ice volume (see Figure 13). There is evidence to support the increase in the drag coefficient for ice covered conductors for two reasons: the first due to the effect of the equivalent diameter and the second due to the ice shape itself as opposed to the round and smooth cylinder.

NOTE The uniform thickness of ice around the conductor corresponds to the minimum overall diameter, i.e. the most compact projected area.

It is assumed that the value of C_i is the same for the ice coverings related to T = 50, 150, 500 years.

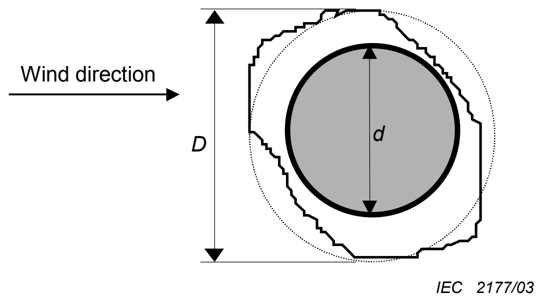


Figure 13 – Equivalent cylindrical shape of ice deposit

6.4.6 Determination of loads on supports

6.4.6.1 Unit action of the wind on the ice-covered conductors

With reference to 6.2.5 the characteristic value (a) of the unit wind action on ice covered conductors with the wind blowing horizontally and perpendicular to the line, is given by the expression:

$$a = q_0 \; C_i \; G_c \; G_L$$

$$q_{oL} = \frac{1}{2} \mu \tau K_R^2 V_{iL}^2 \quad \text{or} \quad q_{oH} = \frac{1}{2} \mu \tau K_R^2 V_{iH}^2 \quad \text{Pa (N/m}^2\text{)}$$

dependent on the loading condition, and with appropriate $C_i = C_{iL}$ or C_{iH}

G_c is the combined wind factor of conductors as defined in 6.2.6.1;

G_L is the span factor as defined in 6.2.6.1;

τ is the density correction factor given in 6.2.5.1.

6.4.6.2 Loads on supports

Two combined wind and ice loading conditions should be considered with their coincident vertical loading.

The load (A_c) in N due to the effect of the wind upon a wind span L, applied at the support and blowing at an angle Ω with the conductors, is given by the following expression, using q_0 of Equation (7).

NOTE 1 The wind span L of a support is equal to half the sum of the length of adjacent spans

$$A_c = q_0\, C_i\, G_c\, G_L\, D\, L \sin^2\Omega$$

For the two recommended loading conditions, the wind force on ice-covered conductors shall be:

– **Condition 1** (highest value of ice load to be combined with average of yearly maximum wind speed during ice persistence):

  $A_{c1} = q_{0H}\, C_{iH}\, G_c\, G_L\, D_L\, L \sin^2\Omega$

  $D_L = (d^2 + 4g_L/9{,}82\pi\delta)^{0{,}5}$

– **Condition 2** (highest value of wind speed during ice persistence to be combined with average of yearly maximum ice load):

  $A_{c2} = q_{0L}\, C_{iH}\, G_c\, G_L\, D_H\, L \sin^2\Omega$

  $D_H = (d^2 + 4g_H/9{,}82\pi\delta)^{0{,}5}$

NOTE 2 These two conditions were found, in general, to be most critical. Should other conditions be required, the information contained in Annex A may be used.

In the above equations, D_L, D_H are diameters (m) of the equivalent cylindrical shapes for the types of ice being considered.

$$g_L \text{ and } g_H = \text{ice load (N/m)}$$

where

δ is the highest density for type of ice being considered (kg/m^3);

Ω is the angle between wind direction and the conductor.

Where support members are critical for lower conductor vertical loads at the supports, the effect of reduced vertical loads and the presence of aerodynamic lift forces should be considered. It is suggested that the lift force per unit length is not likely to exceed 50 % of the drag force per unit length of ice covered conductors.

6.5 Loads for construction and maintenance (safety loads)

6.5.1 General

Construction and maintenance operations are the occasions when failure of a line component is most likely to cause injury or loss of life. These operations should be regulated to eliminate unnecessary and temporary loads which would otherwise demand expensive reinforcing of all supports, especially in ice-free areas.

National regulations and/or codes of practice generally provide minimum safety rules and requirements with respect to public safety.

In addition, construction and maintenance loading cases will be established in this standard as recommended hereafter. The system stress under these loadings shall not exceed the damage limit, and the strength of the supports shall be verified either by testing (see IEC 60652) or by reliable calculation methods.

6.5.2 Erection of supports

The strength of all lifting points and of all components shall be verified for at least twice the static loads produced by the proposed erection method. A factor of 1,5 can be used if the operations are carefully controlled.

6.5.3 Construction stringing and sagging

6.5.3.1 Conductor tensions

The tensions shall be calculated at the minimum temperature allowed for stringing and sagging operations. It is recommended that, in the calculation of loads on the structures, conductor tensions of at least twice the sagging tensions be used for conductors being moved and 1,5 times for all conductors in place.

6.5.3.2 Vertical loads

The extra load applied to the supports shall be calculated from the vertical angles, with the conductor tensions given in 6.5.3.1. The loading shall be applied to the conductor attachment points or conductor stringing points (if different), and shall consider all possible conductor stringing sequences in any combination of load and no load at the several support points that represent the conductor stringing sequence.

6.5.3.3 Transverse loads

Angle structures shall be capable of resisting the transverse loads produced by the conductor tensions given in 6.5.3.1.

Although light winds can occur during construction and maintenance, their effect is neglected for these calculations.

6.5.3.4 Longitudinal (and vertical) loads on temporary dead-end supports

a) Longitudinal loads

   Supports used as dead-ends during stringing and sagging shall be capable of resisting longitudinal loads resulting from the sagging tensions given in 6.5.3.1 in any combination of load and no load at the several support points that represent the conductor stringing sequences.

b) Vertical loads

   If such structures are reinforced by temporary guys to obtain the required longitudinal strength, these guys will increase the vertical loads at the attachment points and shall be adequately pre-stressed if attached to a rigid tower. It will therefore be necessary to check the tension in the guys and take account of the vertical loads applied to the attachment points.

NOTE Pre-stressing is required because of differences in deformation of guys vs. lattice crossarm when both are subjected to load.

6.5.3.5 Longitudinal loads on suspension supports

While the conductor is in the stringing sheaves, a longitudinal load shall be applied to the supports. This load is equal in value to the unit weight of the phase conductor, *w* (N/m), multiplied by the difference in elevation of the low points of adjacent spans (m). This load (in N) will be negligible and much less than the containment loads derived in 6.6.3. except for unusual spans, where it shall be verified that the structure can resist at least twice this load.

In operations such as conductor tie-downs, loads are applied at all conductor points and shall be taken into account.

6.5.4 Maintenance loads

All conductor support points shall be able to resist at least twice the bare conductor vertical loads at sagging tensions.

Temporary lift or tension points, close to the normal attachment points of conductors and used for maintenance or live line operations, shall also be able to resist at least twice the bare conductor loads at sagging tensions.

A factor of 1,5 instead of 2 for the above loads can be used if the operations are carefully controlled.

Those responsible for maintenance shall specify lifting arrangements which will not overstress the structure.

All structural members that may be required to support a lineman shall, by calculation, be able to support a 1 500 N load, applied vertically at their midpoint, conventionally combined with the stresses present during maintenance. These are usually based on still air at the minimum temperature assumed for maintenance operations.

6.6 Loads for failure containment (security requirements)

6.6.1 General

The objective of security measures is to minimize probability of uncontrolled propagation of failures (cascades) which might otherwise extend well beyond the failed section, whatever the extent of the initial failure.

The security measures detailed below provide for minimum security requirements and a list of options which may be used whenever higher security is justified.

The loads prescribed in 6.6.3 provide conventional lattice structures with the means of minimizing the probability of cascade failures. These requirements are derived from experience on conventional lattice structures, but should also be applicable to other types of structures. Service experience using different types of structures or materials could dictate or require different or additional precautions that can be substituted to the above requirements.

The system stress under these loads shall not exceed the failure limit.

6.6.2 Security requirements

Unless special limiting devices are used, the loadings specified in 6.6.3 shall be considered as minimum requirements applicable for most transmission lines.

In cases where increased security is justified or required (for example on very important lines, river crossings or lines subjected to maximum ice loads), additional measures or loadings can be used according to local practice and past experience. A list of such measures appears in 6.6.3.3.

6.6.3 Security related loads – Torsional, longitudinal and additional security measures

6.6.3.1 Torsional load

At any one ground wire or phase conductor attachment point the relevant, if any, residual static load (RSL) resulting from the release of the tension of a whole phase conductor or of a ground wire in an adjacent span shall be applied. This RSL shall be considered at sagging temperatures without any wind or ice loads.

The RSL for suspension structures shall be calculated for average spans and at sagging tensions, allowance being made for the relaxation of the load resulting from any swing of the insulator strings assemblies, deflection or rotation of the structure, foundations, articulated crossarms or articulated supports, and the interaction with other phases conductors or wires that may influence this load.

The value of the RSL may be limited by special devices (slipping clamps, for example), in which case, the minimum security requirements should be adjusted accordingly.

Coincident bare conductor loads at sagging tensions shall be applied at all other attachments points.

6.6.3.2 Longitudinal loads

Longitudinal loads shall be applied simultaneously at all attachment points. They shall be equal to the unbalanced loads produced by the tension of bare conductors in all spans in one direction from the structure and with a fictitious overload equal to the weight, w, of the conductors in all spans in the other direction. Average spans shall be considered with the bare conductors at sagging tension and any appropriate relaxation effects, as mentioned in 6.6.3.1 shall be considered. See Figure 14.

An alternative proposal would be to consider about 50 % of the sagging tension at each attachment point.

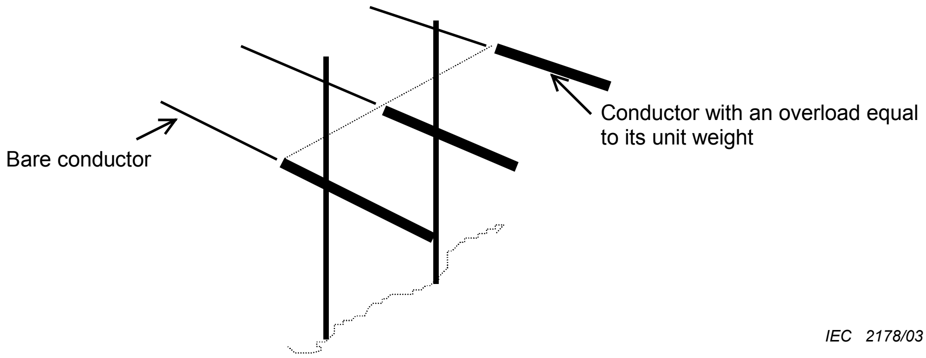


Figure 14 – Simulated longitudinal conductor load (case of a single circuit support)

6.6.3.3 Additional security measures

The designer can increase the security by adopting some of the requirements listed in Table 9.

Table 9 – Additional security measures

Description of additional security measures	Comment
Increase the RSL by a factor of 1,5 at any one point	Lines where higher security is justified
Increase the number of torsional/flexural load points to either two phases or two ground wires where the residual static load (RSL) is applied	Advisable for double or multi-circuit lines
Calculate the RSL for tensions higher than the every day load by using wind or ice load corresponding to a 3 year return period in conjunction with this loading case	Advisable for angle structures or lines subjected to severe climatic conditions
Insertion of anti-cascading towers at intervals, typically every tenth tower. These towers shall be designed for all broken conductors with limit loads	To be considered for important lines in heavy icing areas

7 Strength of components and limit states

7.1 Generalities

The purpose of this clause is to define limit states of line components and their common statistical parameters.

When subjected to increasing loads, line components may exhibit at some load level a permanent deformation, particularly if the failure mode is ductile. This level is called the damage or serviceability limit state. If the load is further increased, failure of the component occurs at a level called the failure or ultimate limit state.

The transmission line is considered intact when its components are used at stresses below their damage limit. It is considered in a damaged state if its components have exceeded their damage limit state. Finally the line is considered to have failed if its components have reached their failure limit. The graphical interpretation is shown on Figure 15.

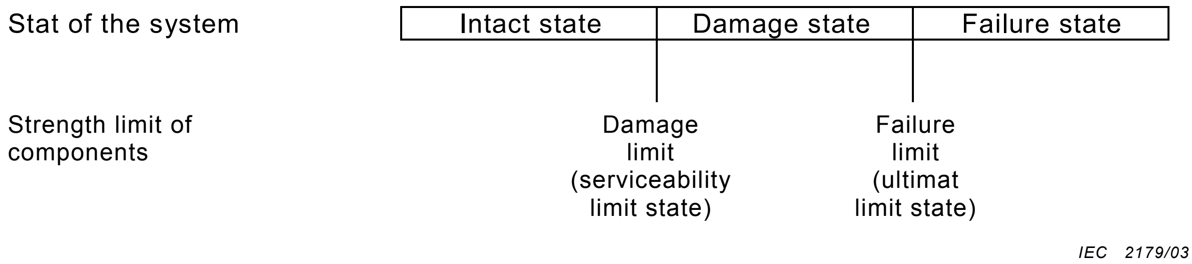


Figure 15 – Diagram of limit states of line components

7.2 General equations for the strength of components

With reference to Equations (3) and (4):

$$(\text{effect of } Q_T) < \Phi_N \times \Phi_S \times \Phi_Q \times \Phi_C \times R_c$$

During design, each component shall satisfy load and strength requirements for reliability, security and safety conditions. In practice, two sets of equations (reliability and safety) determine the damage characteristic strength required for the component, and a third set of equations (security) determines the failure characteristic strength required for the component. In these equations, the reliability conditions shall normally be the governing condition for the main components.

7.2.1 Values of strength factor Φ_N

Whenever a number *N* of components are expected to be subjected to the same critical load Q_T during a single occurrence of a climatic event, the characteristic strength of individual components shall be de-rated (multiplied) by a strength factor Φ_N, This factor depends on N and on characteristics of the strength distribution function (type and coefficient of variation v_R) of strength *R*.

In the absence of specific experience, the number N of supports subjected to the maximum load intensity during a single occurrence of climatic events can be derived from Table 10.

Table 10 – Number of supports subjected to maximum load intensity during any single occurrence of a climatic event

Loading	Flat to rolling terrain	Mountains
Maximum gust wind	1 (1 to 5)	1 (1 to 2)
Maximum ice	20 (10 to 50)	2 (1 to 10)
Maximum ice and wind	1 (1 to 5)	1 (1 to 5)
NOTE Values in brackets represent the typical range of supports based on a span of 400 m.		

The number of components other than supports can be directly derived from the number of supports thus selected.

Values of Φ_N are given in Table 11 and are based on a normal distribution function. In the same table, the values within brackets are based on the log-normal distribution function. Values derived from other distribution functions can be used if more representative of the component being designed.

In the case of high values of v_R and N (see the shaded cells with italic figures in Table 11), the value of Φ_N is very sensitive to the choice of the distribution function. Thus, engineering judgement and strength test results should be used in the selection of the appropriate distribution function. In Table 11 the values outside the shaded area are conservatively taken from the normal distribution curve. Should the strength distribution curve be known, Annex A can be used to provide the specific values for the normal and log-normal distributions.

Table 11 – Strength factor Φ_N related to the number N of components or elements subjected to the critical load intensity

	Coefficient of variation of strength v_r						
N	**0,05**	**0,075**	**0,10**	**0,15**	**0,20**	**0,25**	**0,30**
1	1,00	1,00	1,00	1,00	1,00	1,00	1,00
2	0,98	0,98	0,97	0,94	0,91	0,87	0,84
5	0,96	0,94	0,92	0,85	0,80	*0,72 (0,83)*	*0,64 (0,80)*
10	0,94	0,92	0,89	0,81	*0,72 (0,82)*	*0,62 (0,77)*	*0,51 (0,73)*
20	0,93	0,90	0,85	*0,77 (0,83)*	*0,66 (0,77)*	*0,53 (0,73)*	*0,38 (0,68)*
40	0,92	0,87	0,83	*0,72 (0,80)*	*0,59 (0,74)*	*0,44 (0,69)*	*0,26 (0,64)*
80	0,91	0,86	*0,79 (0,84)*	*0,68 (0,77)*	*0,53 (0,71)*	*0,36 (0,65)*	*0,16 (0,60)*
160	0,90	0,85	*0,79 (0,83)*	*0,67 (0,76)*	*0,52 (0,69)*	*0,34 (0,62)*	*0,13 (0,57)*

7.2.2 Strength factor Φ_S related to the coordination of strength

It is often cost-effective to design some components to be more reliable than others in order to minimize the consequences (i.e. repair time, secondary failure, etc.) of a possible failure due to climatic event.

In order to achieve such strength coordination, a strength reduction factor Φ_{S2} is applied to the strength of components R_2 chosen to be more reliable while a factor Φ_{S1} = 1,0 is applied to the first component to fail. Factor Φ_{S2} depends on the coefficient of variation of both components and is given in Table 12. It is based on a confidence of 90 % that the second component R_2 will not fail before the first R_1. Thus, 90 % is the confidence level on the target sequence of failure.

Table 12 – Values of Φ_{S2}

		COV of R_1			
		0,05	**0,075**	**0,10**	**0,20**
COV of R_2	0,05-0,10	0,92	0,87	0,82	0,63
	0,10-0,40	0,94	0,89	0,86	0,66

NOTE In the above table, R_2 is the component designed more reliable than R_1.

Criteria for deciding on a preferred strength coordination are discussed in Annex A, and a usually accepted strength coordination is given in Table 13. This table provides first for the strength coordination between major components and, subsequently, provides for a subsequent coordination within the various elements of a major component.

Table 13 – Typical strength coordination of line components

	Major component	Coordination within major components *
Less reliable	Suspension tower	Tower, foundations, interfaces
More reliable with 90 % confidence	Tension tower	Tower, foundations, interfaces
	Dead-end tower	Tower, foundations, interfaces
	Conductors**	Conductors, insulators, interfaces

* Within each major component, the underlined component is the least reliable with 90 % confidence.

** With the strength limits specified in Table 16, conductors are usually the most reliable component of the line.

7.3 Data related to the calculation of components

7.3.1 Limit states for line components

Tables 14 to 17 specify damage limits and failure limits for line components with regard to the system. In the absence of relevant data, these values constitute acceptable design limits. If local data and national experience is available, it can be used to improve and complete the tables.

Table 14 – Damage and failure limits of supports

Supports			Damage limit	Failure limit
Type	Material or elements	Loading mode		
Lattice towers, self-supporting or guyed	All elements, except guys	Tension	Yield (elastic) stress	Ultimate (breaking) tensile stress
		Shear	90 % (elastic) shear stress	Shear (breaking) stress
		Compression (buckling)	Non-elastic deformation from l/500 to l/100	Collapse by instability
	Steel guys	Tension	Lowest value of: – yield stress (70 % to 75 % UTS) – deformation corresponding to 5 % reduction in tower strength – need to readjust tension	Ultimate tensile stress
Poles	Steel	Moments	1 % non-elastic deformation at the top, or elastic deformation that impairs clearances	Local buckling in compression or ultimate tensile stress in tension
		Compression (buckling)	Non elastic deformation from l/500 to l/100	Collapse by instability
	Wood	Moments	3 % non-elastic displacement at the top	Ultimate tensile stress
		Compression (buckling)	Non-elastic deformation from l/500 to l/100	Collapse by instability
	Concrete	Permanent or non- permanent loads	Crack opening after release of loads, or 0,5 % non-elastic deformation	Collapse of the pole

NOTE 1 The deformation of compression elements is the maximum deflection from the line joining end points. For elements subjected to moments, it is the displacement of the free end from the vertical.

NOTE 2 l is the free length of the element.

NOTE 3 The width of crack for concrete poles to be agreed upon.

Table 15 – Damage and failure limits of foundations

Foundations			Damage limit	Failure limit
Type	**Support type**	**Statically determinate movement**		
Uplift	Guyed	Yes	Need to readjust tension in guys	Excessive out-of-plane uplift movement (plane formed by the other three foundations) in the order of 5-10 cm
		No	5 % reduction in support strength	
	Self-supporting	Yes	1° (degree) rotation of the support	
		No	Differential vertical displacement equal to *Y*/300 to *Y*/500 with a maximum of 2 cm	
Compression	All types	Yes	Displacement corresponding to a 5 % reduction in the support strength	Excessive out-of-plane settlement (plane formed by the other three foundations) (in the order of 5-10 cm)
		No	Differential vertical displacement equal to *Y*/300 to *Y*/500 with a maximum of 2 cm	
Moments (rotations)	Poles	Yes	2° (degree) rotation of the support	Excessive rotation in the order of 5° to 10°
		No	Rotation corresponding to a 10 % increase in the total moment due to eccentricity	

NOTE 1 Takes into account the interaction between the support and its foundation.

NOTE 2 A determinate movement is one that does not induce internal efforts in the structure. For example the displacement of one foundation of a three-legged support is a determinate movement, while the displacement of a four-legged support is an indeterminate movement.

NOTE 3 *Y* is the horizontal distance between foundations.

NOTE 4 Some rigid foundations (e.g. pile) may require lower limits.

Table 16 – Damage and failure limits of conductors and ground wires

Conductors and ground wires	Damage limit	Failure limit
All types	Lowest of: - vibration limit, or - the infringement of critical clearances defined by appropriate regulations, or - 75 % of the characteristic strength or rated tensile strength (typical range in 70 % to 80 %)	Ultimate tensile stress (rupture)

Table 17 – Damage and failure limit of interface components

Type of interface components	Damage limit	Failure limit
Cable joints – dead-end and junction fittings – suspension fittings	Unacceptable permanent deformation (including slippage)	Rupture
Insulators (porcelain and glass)	70 % strength rating or broken shed (glass only)	Rupture of pin, cap, cement or shed
Fittings	Critical permanent deformation	Rupture of fittings or shear of bolts
NOTE 1 Normally, fittings are designed in a manner to reduce or eliminate wear. Should wear be expected because of point-to-point contact, it should be considered in the design. In such case, the damage limit becomes 'exceeding the excepted wear'. NOTE 2 The critical permanent deformation is defined as the state where the fittings cannot be easily taken apart.		

7.3.2 Strength data of line components

For practical considerations, it is assumed that the normal density function is adequate for the statistical distribution of the strength of line components. As indicated earlier, log-normal density function can also be used to characterize strength variation, mainly for components with brittle behaviour or subjected to stringent quality control.

This assumption of normal density function is quite true for many line components, particularly those having a low coefficient of variation.

If no specific tests are available, the characteristic strength R_c will be found in ruling standards; R_c may be assumed to correspond to $e = 10\ \%$. Table 18 gives typical strength coefficient of variation v_R to be used as default value in the absence of relevant data.

If tests are available, $R_c = (10\ \%)$, $R = (1 - u \times v_R)\ \overline{R}$; if R is assumed normally distributed, $u = 1{,}28$, or given in Table 19 for log-normal distribution function.

NOTE The value of $u = 1{,}28$ corresponds to a large number of samples. For fewer samples, different values derived from statistical properties of the normal distribution function can be used

Table 18 – Default values for strength coefficients of variation (COV)

Component	COV
Conductors and ground wires (strength usually limited by joints)	0,03
Fittings	0,05
Insulators	0,05
Steel poles	0,05
Concrete poles	0,15
Wood poles	0,20
Lattice towers	0,10
Grouted rock anchors	0,10
Pile foundation	0,25
Foundation with undercut or machine-compacted backfill	0,20
Foundation with uncompacted backfill	0,30

Table 19 – *u* factors for log-normal distribution function for *e* = 10 %

COV	*u*
0,05	1,26
0,10	1,24
0,20	1,19
0,30	1,14
0,40	1,08

7.3.3 Support design strength

Supports shall be designed for a characteristic strength R_c equal to:

$$R_c \geq \frac{\text{Support design loads}}{\Phi_N \, \Phi_S \, \Phi_Q \, \Phi_C}$$

Structure design loads comprise the dead loads and external loads.

Φ_N is selected according to 7.2.1.

Φ_S is derived from Table 12. It is equal to 1,0 if the support is selected as the least reliable component. Note that it may be advisable to design tower parts such as crossarms and ground wire peaks, with a sub-sequence of failure within the tower so that failure of these parts will not cause failure of main tower body.

Φ_Q for lattice towers, Table 20 gives recommended values for Φ_Q, to take into account the quality in calculation method, fabrication and erection. For other supports, coefficients Φ_Q of the same order can be estimated by view of local conditions.

Φ_C can be taken equal to 1,0, especially when the characteristic strength corresponds to a 10 % exclusion limit. If the exclusion limit varies greatly from 10 %, refer to Annex A for possible adjustments.

Table 20 – Value of quality factor Φ_Q for lattice towers

Quality control	Φ_Q
Very good quality control such as involving third party inspection	1,00
Good quality control	0,95
Average quality control	0,90

Structures subjected to full scale (type) tests shall withstand loads equivalent to R_c. Tests shall conform to the latest version of IEC 60652.

7.3.4 Foundation design strength

The maximum reactions on foundations are obtained from the design of structures subjected to the loads defined in this standard using conventional methods of analysis and appropriate wind-weight span combinations, tower legs and body extensions. The reactions thus obtained are considered to be the design loads on foundations. When foundation tests are required, these shall be performed in accordance with the latest version of IEC 61773.

The characteristic strength of foundations R_c, shall meet the following requirement:

$$R_c \geq \frac{\text{Foundation design loads}}{\Phi_N \, \Phi_S \, \Phi_Q \, \Phi_C}$$

Φ_N depends on the number of foundations subjected to maximum load intensity in a given storm event. For example, if N = 2, and COV = 0,20, Φ_N = 0,91 can be obtained from Table 11.

Φ_S can be obtained from Table 12, based on the expected strength COV. For default COV values, refer to Table 18.

If characteristic strength R_c is derived from tests typical of actual line construction, then Φ_Q = 1. However, if foundation tests were carried out in a controlled environment not typical of line construction, then it is suggested to consider Φ_Q = 0,9.

Φ_C can be taken equal to 1,0, specially when the characteristic strength corresponds to a 10 % exclusion limit. This is usually the case when R_c is deducted from foundation tests. In case the exclusion limit varies greatly from 10 %, refer to Annex A for possible adjustments.

7.3.5 Conductor and ground wire design criteria

Conductors and ground wires are designed for the most critical resultant load(s) per unit length applied to the corresponding ruling span.

In this case, $\Phi_N = \Phi_S = \Phi_Q$ = 1,0 and the maximum conductor tension shall not exceed R_c as defined in Table 16.

When required, conductor tests shall comply with the latest version of IEC 61089.

7.3.6 Insulator string design criteria

The calculation of the insulator strings is based on their relationship to the conductors to which they are attached. These are dealt with in the same way as for the support/foundation relationship. The critical design loading shall be derived from the maximum calculated conductor loading to which the components are attached.

Φ_N shall be derived in accordance with Table 11.

Φ_S = Φ_{S2} = 0,90 for all insulator strings, for which the COV generally remains under 7 % (see Tables 12 and 17).

Φ_C = 1,0, and Φ_Q = 1,0 (unless poor quality material).

In addition to the above requirements, it is advisable, particularly for countries subjected to ice loads, to select the characteristic strengths of dead-end insulators at least as high as the characteristic strength R_c of attached conductors. Similarly, it is advisable to design the dead-end fittings to withstand, at failure, about 15 % more than the conductor characteristic strength R_c. When required, tests for fittings shall comply with the latest edition of IEC 61284.

Annex A
(informative)

Technical information

A.1 Relations between load and strength

A.1.1 Estimate of line reliability

A.1.1.1 Combinations of probabilities

The reliability of a system[1] is a function of the reliability of its components. When these components are in series, the yearly reliability of the system (P_{ss}) is equal to the product of the yearly probabilities of survival of the individual components (P_{si}):

$$P_{ss} = P_{s1}\, P_{s2} \ldots\, P_{sn} = \prod_{i=1}^{n} P_{si} \qquad \text{(A.1)}$$

In cases where the unreliability of components is smaller than 10^{-2}, which is usually the case with typical transmission lines, and where the least reliable component has a failure probability or a rate of unreliability which is approximately one order of magnitude higher than that of other components, the reliability of the system can be approximated by that of the least reliable component.

This situation occurs naturally in many transmission lines. For example, in non-icing areas, conductor tension limitations of every day stress to obviate vibration problems may limit the maximum tension under wind to about 50 % of the tensile strength of the conductors. This in turn increases very substantially the probability of survival of conductors as compared to supports. Consequently, conductors in non-icing areas properly installed and protected from vibration damage are naturally more reliable than other components.

On a theoretical basis, the reliability (probability of survival) of a transmission line can be estimated using the following procedure.

- For each type of climatically produced load, the probability density function of load f_Q, is first established. This function is adjusted to reflect the maximum loading intensity that might occur within the space covered by the line. Any proven directional tendencies which might affect load intensity may also be weighted in the load function f_Q, otherwise the load is assumed to act in the most critical direction.
- Then the probability density function of strength, f_R, of the line as a system is established. The function can be complex except when lines are designed with a strength coordination approach (see A.1.3.1). In this case, function f_R can be approximated by the strength density function of the weakest component. It should be mentioned that both the load density and the strength density function must refer to the same critical action, e.g. the compression force of the highest strained member and its buckling strength. Practically

1 Additional information and background data on this annex as well as on the following annexes related to reliability based design of overhead lines can be found in the following CIGRÉ publications and brochures: Brochures 109 and 179, *Electra* papers 1991-137 and 2000-189.

speaking this corresponds to having both load and strength expressed in the same units: such as forces (kN) or stresses (N/mm^2). If climatic loads are expressed in terms of wind speed, the load side of the equations can be expressed as “effects of wind speed”.

For a better practical evidence, it is preferable to use the cumulative distribution function of strength, F_R, instead of the density function f_R. The cumulative function F_R is given by the equation

$$F_R(x) = \int_0^x f_R(\xi) d\xi \qquad \text{(A.2)}$$

- In the next step the relative position of the two curves f_Q and F_R has to be defined.

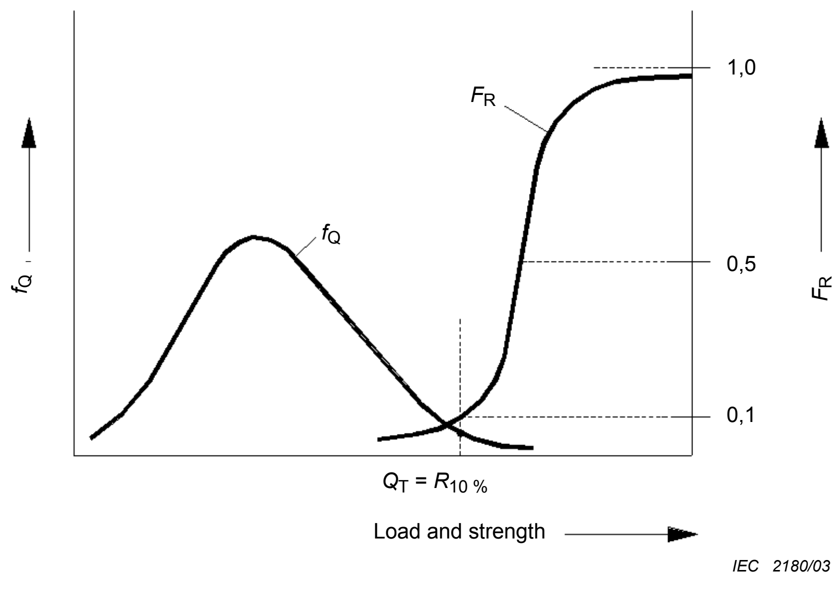
  This relative position can be found through a relation such as: a load with a yearly probability of occurrence $1/T$ is set equal to the strength with a 10 % exclusion limit, or analytically:

$$Q_T = (10\ \%)\ R \qquad \text{(A.3)}$$

The choice of the return period T of load Q_T depends on the desired degree of reliability.

It can be demonstrated that equation (A.3) leads to an almost constant reliability between $(1\text{-}1/T)$ and $(1\text{-}1/2T)$, independent of the shape of the load and strength curves and the coefficients of variation of each variable (see A.1.2.3).

Figure A.1 depicts relative positions of load density and cumulative strength. The positions of the two functions are chosen to comply with equation (A.3). The functions f_Q and F_R are arranged such that the load Q_T having a selected return period T is equal to the strength with an exclusion limit of 10 % (F_R = 0,1).



Key

f_Q probability density function of yearly maximum loads

F_R cumulative distribution function of strength

Q_T load having a selected return period T, e.g. 50 years

Figure A.1 – Relations between load and strength

In practice, the reliability resulting from the relationship expressed in Equation (A.3), can be considered as a minimum value and calculations can be further refined through the introduction of correction factors related to the following items:

- **use factors of components:** the fact that all components are not used at their maximum design parameter (wind span, weight span, height of support, line angle) contributes to an increase of the reliability;
- **characteristic strength:** in actual lines, the characteristic strength of most components corresponds to an exclusion limit less than 10 %. If it is assumed to be equal to 10 % in case of design, than the resulting reliability of the line will be higher;
- **strength coordination:** a selected strength coordination results in an increase of strength or withstand resistance of some components;
- **number of components subjected to maximum loading intensity**: whenever a storm or severe icing occurs, not all structures will be subjected to maximum loads, since the storm is limited in spatial extension;
- **quality control during fabrication and construction:** by these measures low quality material will be eliminated. No components with strengths below a certain limit will be used;
- **wind direction:** in case of wind loads it is assumed that maximum wind velocities also act in the most unfavourable direction. However, maximum winds are distributed in angle sectors. Approximate calculations carried out by the CIGRE Working Group SC 22.06 showed that more realistic assumptions could reduce the probability of failure by one order of magnitude.

While the above-mentioned factors contribute to the actual reliability usually being higher than the theoretical values, other factors could lead to opposite effects, i.e. a reduction in reliability. The ageing of some line components and the fatigue by a large number of loading cycles will have a negative effect on reliability.

It is noted that the above probability of failure is only one of the components of the total line unavailability as described in 3.1.22.

A.1.1.2 Loadings

Loadings on transmission lines can be separated into three groups: external loads, dead loads and special loads.

- **External loads:** External loads are random loads due to wind and ice, taken separately or combined together. Occurrence and magnitude of external loads can be modelled by appropriate statistical functions.

  When statistics of annual maximum values of ice or wind are available, it is commonly accepted that these climatic variables follow an extreme distribution function such as Gumbel type I distribution as described in Clause C.4. For particular cases where ice or snow accretions are not observed each year, some recent studies also suggest that threshold methods and a generalized Pareto distribution may be used. The Gumbel type I distribution function can be defined using two parameters: the mean value and the standard deviation of the applicable variable. However, mean value and standard deviation have to be acquired by evaluation of measurements carried out over a certain period of years. The number of years with observations affects the distribution as well, in particular regarding prediction of events having a long return period *T*.

- **Dead loads:** Dead loads are loads due to the dead weight of support, conductors and insulator strings. Although they are permanent in nature, dead loads vary from one support to another due to variation of support height and weight span of conductors.

  Reliability of lines subjected only to dead loads should be practically 100 %. This reliability is provided by the safety requirements where it is specified that support should be designed for twice the vertical dead load of conductors with insulator strings. These requirements cover the increase in vertical loads during construction and maintenance that occurs during operations on conductors such as stringing, lifting or lowering.

  The dead weight of conductors maintained by supports is the product of bare weight per unit length, which is constant, and the weight span which may vary according to the support spacing, and difference of elevation, and conductor tension, which varies with conductor temperature. In all cases, the weight span of a given support type has a maximum assigned value which is the one considered for the design of supports. There is also a minimum value for uplift considerations of suspension insulator strings and uplift of foundations.

  Since all supports of a transmission line are usually not used at their maximum design spans for weight and wind loads, this variation of spans, if neglected, contributes to an increase in reliability. For evaluation purposes the ratio of actual span to maximum span is defined as being the span use factor U (see Clause B.4).

  Variable U was modelled so that its influence in reliability calculations through appropriate correction factors γ_u, can be applied to support loads.

  Sometimes, supports are designed prior to knowing the span distribution or in other cases the same supports could be used in future projects. In cases where data on use factors are not available or cannot be predicted, the influence of U can be neglected and γ_u is considered equal to 1,0. This will result in more reliable structures. In other cases where data on span use factor are available or can be predicted, the methodology appearing in Clause B.4 can be used in order to achieve some line investment reductions.

- **Special loads:** Special loads consist of external loads that might occur during line construction and maintenance as well as longitudinal and vertical loads provided as a security measure for the prevention of cascading failures. If the external loads exceeded the limit loads along the total line, or a long section of the line, then the provision of special loads would not prevent cascades to occur in these sections.

  Construction and maintenance loads are treated in a deterministic manner and are considered constant. They are established in such a way that their magnitude is not likely to be exceeded under normal construction and maintenance operations. If the magnitude of special loads is such that they are more critical than loadings described in **'external loads'** and **'dead loads'**, overall reliability of the line will be altered.

A.1.1.3 Strength

- **Distribution functions of strength of components**: The strength of line components designed for the same conditions and manufactured accordingly is not constant, as can be observed during routine, sample and type testing. Therefore, the strength has to be treated statistically. The strength of line components can be statistically described either by a normal or a log-normal distribution. The normal (Gaussian) distribution is appropriate for most ductile materials, while the log-normal distribution applies more to brittle materials. With stringent quality control, the distribution function of components may be altered and tends to be a log-normal function. Unless otherwise substantiated by relevant data, the normal distribution can be assumed adequate for most line components so far as the determination of the overall line reliability or designing lines in accordance with the proposed methodology are concerned.

- **Components or elements in series:** When a component is made up of a series of elements, its strength distribution function tends to be an extreme function (minima). With an increasing number of elements in series, both the mean and the standard deviation of the strength of the series are reduced and the resulting distribution function can be approximated by an extreme distribution.

  The statistical parameters of the strength of a series of N components or elements can be derived using available statistical methods. A description of these methods and a derivation of the correction factor Φ_N due to the number of components or elements subjected to maximum load intensity are given in A.1.3.5.

- **Characteristic strength R_c:** This strength is also called guaranteed value or is simply the value specified in relevant standards. In order to establish the characteristic strength, two cases are considered:

  – specific strength tests are performed on the components or elements to be used in a certain line;

  – no such tests are performed.

- **Without tests:** Most components are specified in national standards based on nominal, minimum or guaranteed strength. This is also the case for supports designed with a proven method of calculation based on the minimum mechanical characteristics of the elements.

  The strength of a component is usually set so that these minimum requirements are met by the majority of the components. Recent studies, as well as the analysis of a large number of test results, indicate that the probability of line components not meeting the specified strength (i.e. the exclusion limit) is usually between 1 % and 10 %, therefore the exclusion limit e varies within these limits.

  Consequently, whenever the specified strength of a component, without reference to its exclusion limit e, is taken as the characteristic strength, the assumption of e = 10 % for the characteristic strength of this component leads to higher reliability of the line, or simply to conservative results.

- **With tests:** Whenever tests are performed to determine the strength of a component and its statistical distribution, the characteristic strength can be obtained from the test results after having calculated the mean value $\overline{R}$ and the coefficient of variation $v_R = \sigma_R / \overline{R}$, where σ_R is the standard deviation..

  The strength R according to an exclusion limit e (%) can be obtained from the statistical distribution. For the Gaussian distribution (see C.2.1) the relation is:

$$R_{e\,\%} = \overline{R}(1 - u_e\, v_R) \qquad \text{(A.4)}$$

The factor u_e is the variable of the Gaussian distribution $F_R(u)$ that corresponds to the exclusion limit e. It is the number of standard deviation of the variable R below the mean value R.

$$F_R\ (u_e) = e\ (\%)/100 \tag{A.5}$$

Result

For e = 2 %, u_e = 2,054

e = 5 %, u_e = 1,645

e = 10 %, u_e = 1,282

Therefore, a 10 % probability of being not achieved corresponds to a value of 1,28 standard deviations below the mean value:

if $$R_c = (10\ \%)\ R, \tag{A.6}$$

then $$R_c = \overline{R}\ (1 - 1{,}28\ v_R) \tag{A.7}$$

The value of 1,28 assumes a normal distribution and an infinite number of samples. In practice, it may be used if $N > 10$. For a reduced number of samples, other values could be used to account for statistical uncertainty, as indicated by accepted statistical techniques.

A.1.2 Calculation of reliability

A.1.2.1 General

Load Q and strength R of transmission line components are random variables having each their specific distribution functions. Through technical studies (Clause B.2), it has been recognized that ice and wind variables may be represented by an extreme type I function (Gumbel distribution) while strengths of transmission line components generally follow normal or log-normal functions.

When statistical parameters of load and strength are known, it is possible to calculate or estimate the yearly reliability or probability of survival, P_s, through analytical models or approximate methods. In the following relations, F_Q and F_R are defined as the cumulative distribution functions (CDF) of Q and R while f_Q and f_R are the probability density functions (PDF) of the same variables, e.g. the load of a support leg member and its strength.

The yearly reliability is:

$$P_s = P\ (R - Q > 0) = \int_0^\infty f_Q\ (x)\ F_R\ (x)\ dx \tag{A.8}$$

The value of P_s can be easily obtained by computer programs if f_Q and F_R curves are known. When only the mean and standard deviation of R and Q variables are known, P_s can be obtained from Equation (A.8) as well by assuming the Gumbel distribution for the loads and the Gaussian law for the strengths. However, an estimate of P_s, this is usually called first order analysis, can be obtained in each case through either of the following methods, with F_N being the normal cumulative distribution function:

$$P_s = F_N\ (+\ \beta) \tag{A.9}$$

where

$$\beta = \frac{\overline{R} - \overline{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \quad \text{(A.10)}$$

or

$$\beta = \frac{\ln\left(\overline{R} / \overline{Q}\right)}{\sqrt{v_R^2 + v_Q^2}} \quad \text{(A.11)}$$

If the load *Q* follows a Gumbel distribution and the strength *R* a Gaussian function then

– the format (*R* – *Q*) of Equation (A.10) gives good results for all values of v_Q and when v_R >0,15 and

– the format log-normal (*R*/*Q*) of Equation (A.11) is acceptable when $v_Q = 0,2$ and $v_R \leq 0,15$.

However, when the mean value(s) and standard deviation(s) of *Q* and *R* are known, it is also acceptable to assume a Gumbel function and a Gaussian distribution, respectively, and proceed with these assumptions to calculate the probability of survival P_s using Equation (A.8).

In addition to the above methods, another technique has been found to give results very close to the theoretical ones: it consists of representing the load curve f_Q and the strength curve f_R by normal distributions by choosing the parameters $\overline{Q}$, σ_Q and $\overline{R}$, σ_R such that the upper tail of the load curve f_Q and the lower tail of strength curve f_R match well the given distributions. Then Equations (A.10) and (A.11) can be used with the parameters of the adjusted distributions to determine the yearly reliability.

This tail adjustment format is acceptable for all typical values of v_Q and v_R.

A.1.2.2 Combinations of load and strength

Reliability depends on the parameters of load *Q* and strength *R*. For possible combinations, four cases are considered, each one corresponding to different assumptions:

– Case 1: load *Q* and strength *R* have constant values (Figure A.2a).

– Case 2: load Q is constant and strength *R* is a statistically distributed variable (Figure A.2b).

– Case 3: load *Q* is a statistically distributed variable and strength R is constant (Figure A.2c).

– Case 4: load *Q* and strength *R* are statistically distributed variables (Figure A.2d). This is obviously the most general case and typical of transmission lines.

Table A.1 – Yearly reliability corresponding to various assumptions of load and strength

Case	Load *Q*			Strength *R*			Reliability P_s
	Mean value	**COV**	**Design load**	**Mean value**	**COV**	**Design strength**	
1	Q	0	Q	R	0	$R > Q$	1,0
2	Q	0	Q	$\overline{R}$	V_R	$\overline{R}\,(1\text{-}u_e v_R)$	$1-\int_{-\infty}^{R_c} f_R dL =$ $1\text{-}F_R\,(u_e) = 0{,}90$ (for $u_e = 1{,}28$)
3	$\overline{Q}$	v_Q	Q_T	R	0	R	$1-\int_{Q_T}^{+\infty} f_Q dL = 1-1/T$
4	$\overline{Q}$	v_Q	Q_T	$\overline{R}$	V_R	$\overline{R}\,(1\text{-}u_e v_R)$	$1-\int_{-\infty}^{+\infty} f_Q F_R dL \approx 1-1/2T$

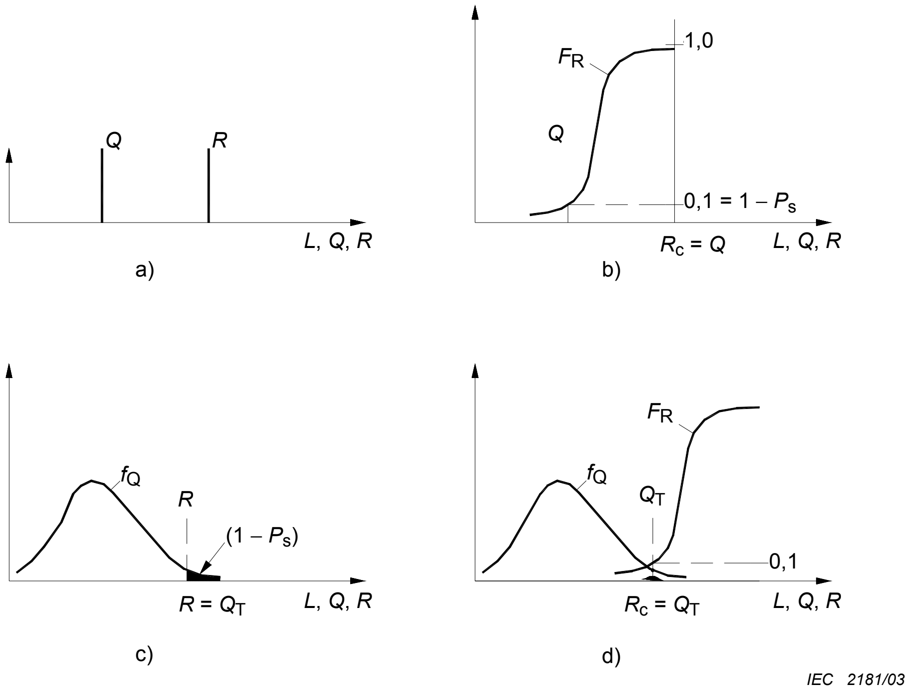
– Case 1: Since load Q and strength R are constant and strength R is greater than load Q the reliability is one or 100 %.

– Case 2: Since load Q is constant the design strength R_e % according to the exclusion limit e is equal to the constant load Q, the yearly reliability P_s is equal to F_R (u_e) and is 0,90 if the exclusion limit for strength is 10 %. P_s is the probability that load Q will be less than strength R.

– Case 3: Since strength R is constant and equal to the load Q_T the yearly reliability depends only on the return period T (Figures A.3 to A.5). The reliability is (1 – 1 / T).

– Case 4: Since load Q and strength R are statistically distributed variables and matched by the relation $\overline{R}(1 - u_e v_R) = Q_T$ the yearly reliability depends on the parameters of the load and the strength side. For COV of 0,20 to 0,50 on the load and 0,05 to 0,20 on the strength side the reliability depends on these data. As can be seen from Figures A.3 to A.5, the reliability can be approximated by 1 – 1 / 2T for COV of strength of 0,10 that applies to supports of overhead lines.

The yearly reliabilities given in Table A.1 apply for design with an exclusion limit of 10 %.



a) case 1, load Q and strength R are constant, $R > Q$

b) case 2, load Q is constant and strength R is variable $\overline{R}(1 - u_e v_R) = Q$

c) case 3, load Q is variable and strength R is constant $R = Q_T$

d) case 4, load Q and strength R are variables $\overline{R}(1 - u_e v_R) = Q_T$

f_Q probability density function of yearly maximum loads

F_R cumulative distribution function of strength (usually support)

Q_T load corresponding to return period T

Figure A.2 – Relations between loads and strengths

A.1.2.3 Load-strength relations

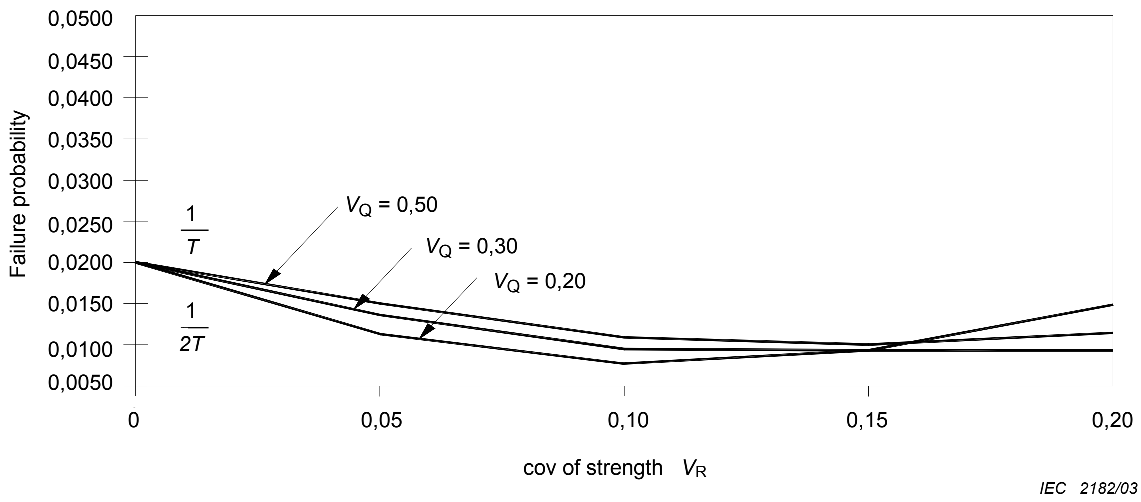
A major breakthrough in probability methods occurred when a relation between load and strength that leads to an almost constant probability of failure was established. This relation consists of associating a load having a return period T with the strength having an exclusion limit of 10 % (or met with 90 % probability). It can be expressed as follows:

$$Q_T = (10\ \%)\ R \text{ or } Q_T = R_{e\ =\ 10\ \%} \qquad \text{(A.12)}$$

Equation (A.12) was found to give a consistent reliability P_s typically in the range of $(1 - 1/T)$ to $(1 - 1/2T)$, with $P_s \sim (1 - 1/2T)$ with the most frequent values for v_Q and v_R. These results remain valid for various distributions of load curves Q such as extreme type I (Gumbel), log-normal and Frechet as well as for normal and log-normal distribution of strength R.

Calculations of failure probabilities covering the most common combinations of Q and R are shown in Figures A.3, A.4 and A.5. The range of v_Q from 0,20 to 0,50 simulates respectively a wind speed variation of 0,10 as well as an ice load variation of 0,50, while the range of v_R of 0,05 to 0,20 covers the variation of the weakest component of the line, usually the supports, where $0{,}05 \le v_R \le 0{,}10$. Figure A.3 applies for T = 50 years, Figure A.4 for T = 150 years, and Figure A.5 for T = 500 years.

The probability of failure calculated when strength and load have dispersions different from the above assumptions may lead to different results.

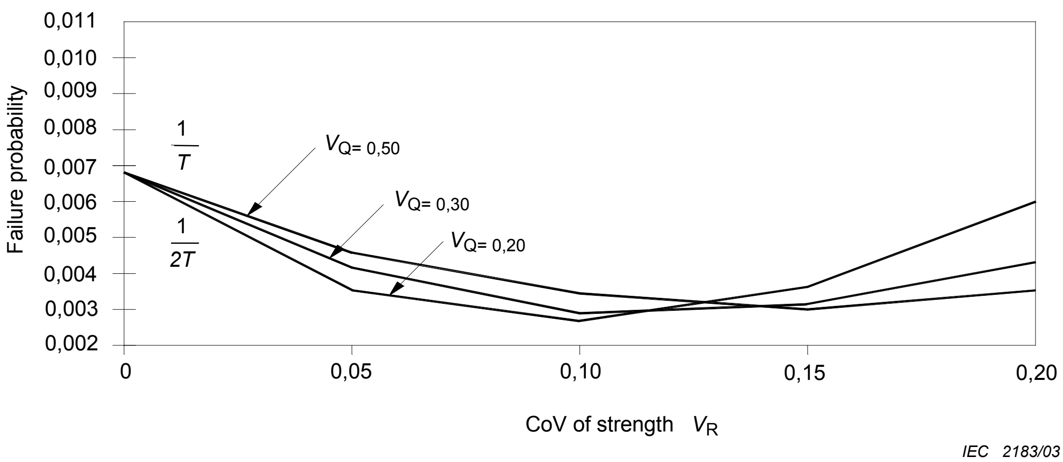


If $v_R = 0,0$ $P_f = 0,02 = 1/50 = 1/T$

If $0,05 < v_R < 0,15$ $P_f \approx 0,01 = 1/100 = 1/(2T)$

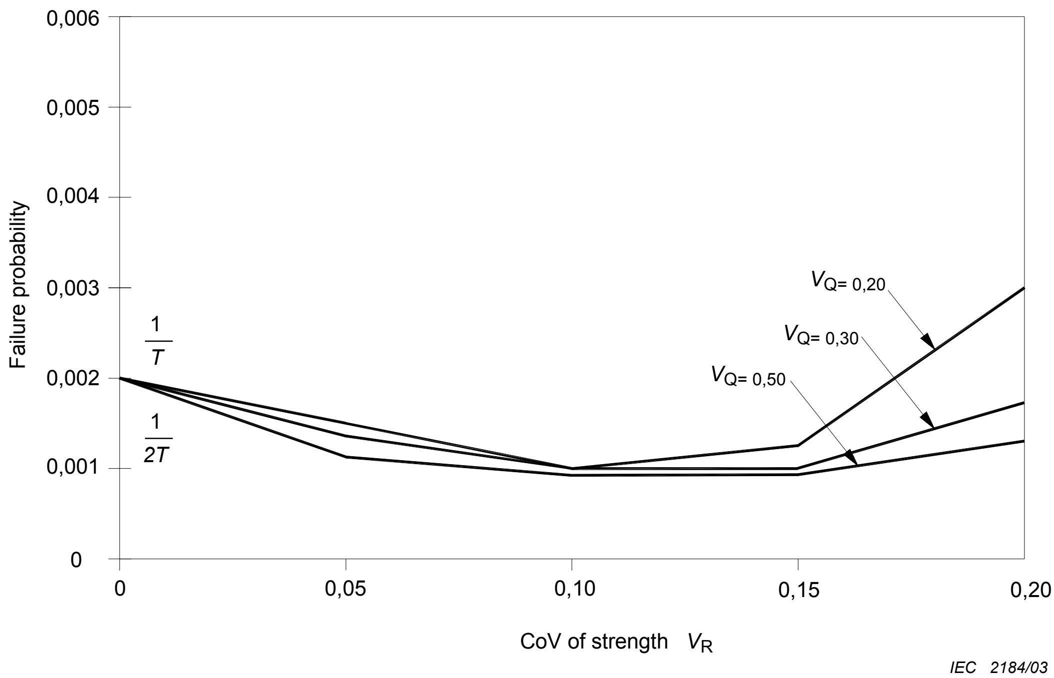
NOTE Distribution of Q is extreme type I and of R is normal.

Figure A.3 – Failure probability $P_f = (1 – P_s)$ for various distributions of Q and R, for T = 50 years



NOTE Distribution of Q is extreme type I and of R is normal.

Figure A.4 – Failure probability $P_f = (1 – P_s)$ for various distributions of Q and R, for T = 150 years



NOTE Distribution of Q is extreme type I and of R is normal.

Figure A.5 – Failure probability P_f = (1 – P_s) for various distributions of Q and R, for T = 500 years

When further examining Equation (A.12), more advantages can be seen of this format.

Firstly, loads are specified using a set of return periods, an approach very common in building design and in weather related loads.

Secondly, the strength corresponding to load Q_T is the one having a 90 % probability of being met. From the analysis of strength data, it is found that the characteristic strength always falls below the 10 % exclusion limit. In cases where strength data are scarce, the reliability calculated using Equation (A.12) can be viewed as a lower boundary of the real value and leads to designs on the conservative side.

Thirdly, if exclusion limits smaller than 10 % are considered (e.g. 2 %), this leads to over-estimating the reliability P_s in cases where the characteristic strength cannot meet the 98 % requirement.

A.1.2.4 Reliability levels and associated failure probabilities

Three reliability levels are generally proposed for line design. They are characterized by the return period of 50, 150 and 500 years for the climatic limit loads. As can be seen from the examples given in A.1.2.3 the theoretical yearly failure probability P_f is between $1/T$ and $1/2T$ depending on the strength variation, where T is the return period of the climatic event under discussion. The yearly probability of survival or the yearly reliability is then

$$P_{s1} = 1 - P_{f1} \tag{A.13}$$

From this basic consideration the failure probability and the probability of survival P_{sN} for a life cycle of a line of N years can be determined by

$$P_{sN} = P_{s1}^{N} = [1 - P_{f1}]^{N} \tag{A.14}$$

and the failure probability P_{fN} for this period can be determined from $P_{fN} = 1 - P_{sN}$.

Table A.2 gives values of probabilities of survival and failure corresponding to a 50 year life cycle for a line.

For a return period of loads of 500 years the theoretical yearly failure probability is between 0,002 and 0,001, however that failure probability related to a life cycle of 50 years would be 0,05 to 0,10.

The examples show that there is a theoretical probability of failure during the life cycle period of a line which is not negligible even for return periods of limit loads of 500 years and more.

It is however important to recognize that exceeding a design load does not necessarily lead to a loss of reliability because the latter is a combination of an extreme load event and its occurrence on a component which is unable to resist its effect. This explains the discrepancy between the probability of failure and that of exceeding a load event.

Table A.2 – Relationship between reliability levels and return periods of limit loads

Return period of limit loads, *T*		**50**	**150**	**500**
Yearly minimum reliability	P_{s1}	0,98 to 0,99	0,993 to 0,997	0,998 to 0,999
Yearly failure probability	P_{f1}	0,02 to 0,01	0,0067 to 0,0033	0,002 to 0,001
Reliability during 50 years life cycle	P_{s50}	0,36 to 0,61	0,71 to 0,86	0,90 to 0,95
Theoretical probability of failure during 50 years life cycle	P_{f50}	0,64 to 0,39	0,29 to 0,14	0,10 to 0,05

A.1.2.5 Selection of reliability levels

Transmission lines can be designed for different reliability levels (or classes) depending on local requirements and the line duties within a supply network.

Designers can choose their reliability levels either by calibration with existing lines that have had a long history of satisfactory performance or by optimisation methods found in technical literature.

In all cases, lines should at least meet the requirements of a reliability level characterized by a return period of loads of 50 years (level 1).

An increase in reliability above this level could be justified for more important lines of the network as indicated by the following guidelines:

It is suggested to use a reliability level characterized by return periods of 150 years for lines above 230 kV. The same is suggested for lines below 230 kV which constitute the principal or perhaps the only source of supply to a particular electric load (level 2).

Finally, it is suggested to use a reliability level characterised by return periods of 500 years for lines, mainly above 230 kV which constitute the principal or perhaps the only source of supply to a particular electric load. Their failure would have serious consequences to the power supply.

The applications of the reliability for overhead lines, including corresponding voltage levels, may be set differently in individual countries depending on the structure of the grid and the consequences of line failures. The impacts on other infrastructure installations such as railroads and motorways should be considered as well.

When establishing national and regional standards or specifications, decisions on the reliability level should be made taking into consideration also the experience with existing lines.

A.1.3 Strength coordination of line components

A.1.3.1 General basis for strength coordination

Transmission line components have different strength variations and responses to loading. When subjected to given loads, failure of components in series could occur whenever load exceeds strength in any component.

In order to decide on an appropriate strength coordination, the following criteria constitute a consensus within the overhead line industry:

a) The first component to fail should be chosen so as to introduce the least secondary load effect (dynamic or static) on other components in order to minimize the probability of a propagation of failure (cascading effect).
b) Repair time and costs following a failure should be kept to a minimum.
c) The first component to fail should ideally have a ratio of the damage limit to the failure limit near 1,0. It should be mentioned that it might be difficult to co-ordinate the strength of components when the least reliable one has a very large strength variation.
d) A low cost component in series with a high cost component should be designed to be at least as strong and reliable as the major component if the consequences of failure are as severe as failure of that major component. An exception of this criterion is when a component is purposely designed to act as a load limiting device. In such a case its strength has to be well tuned with the component it is supposed to protect.

If line components such as suspension supports, tension supports, conductors, foundations and insulator strings are analysed using the above criteria, it can be concluded that:

– conductors should not be the weakest component because of a), b) and c);
– fittings because of d);
– tension support because of a) and b);
– and foundations because of b) and c).

The logical consequence of the considerations above is that the suspension supports should constitute the component with the lowest strength. When a line designed according to this rule is subjected to climatic loads exceeding design values the suspension supports would fail first.

Table A.3 represents a typical strength coordination which takes care of the criteria described above.

Table A.3 – Typical strength coordination

	Major components	**Coordination within major components***
Lowest strength	Suspension tower	<u>Tower</u>, foundations, fittings
Not having lowest strength with 90 % confidence	Tension tower, or Dead end tower, or Conductors	<u>Tower</u>, foundations, fittings <u>Tower</u>, foundations, fittings <u>Conductors</u>, insulators, fittings
* Within each major component the underlined component is the weakest with 90 % confidence.		

While these criteria are the result of logical deduction, it is found in practice that, with a few exceptions, the design of the great majority of existing transmission lines conforms closely to the proposed coordination of strength.

Furthermore, it should be noted that the coordination of strength, as mentioned in A.1.3, also simplifies reliability calculations.

The above strength coordination can be applied to most lines. However there will be some situations where different criteria could be used and thus lead to another sequence of failure.

For example, special river crossing supports could be designed stronger than the conductors. In avalanche areas, where construction of supports is very difficult, the conductor may also be chosen as the weakest component, provided that suspension supports are designed for the forces resulting from the failure of the conductors. Otherwise the failure of conductors would very probably lead to the failure of adjacent supports.

A.1.3.2 Methods for calculating strength coordination factors Φ_S

In order to develop strength coordination factors Φ_S leading to the target strength coordination two methods can be considered:

– Use of different exclusion limits

  For the weakest component, use limit loads in conjunction with 10 % exclusion limit (as suggested in this approach). The next weakest components will be designed with a lower exclusion limit (say 1 % to 2 %), corresponding to the same limit loads.

– Design for a target confidence level in the strength coordination

  Strength coordination factors have to be established in such a way that the target strength coordination between two components, as mentioned in Table A.3, will be reached with a high level of confidence (nearly 80 % to 90 %).

However, due to the random nature of strength, it is theoretically impossible to guarantee with 100 % confidence that the planned coordination of strength will be met in all cases.

A.1.3.3 Use of different exclusion limits

In this method, the confidence level in obtaining the envisaged sequence of failure is variable and depends on strength variations of components. For example assume:

Return period:	T = 50 years;
Mean value of load:	$\overline{Q}$ = 1,0;
Coefficient of variation of load:	v_Q = 0,20;
Coefficient of variation of strength:	$v_{R1} = v_{R2}$ = 0,10.

(R_1 and R_2 represent the strength distribution of respectively the weakest and the next weakest component.)

Both components are designed for a load Q_{50} = 1,52 $\overline{Q}$ (from Gumbel distribution see Table B.1).

The corresponding strength of component R_1 should be (10 %) R_1. For component R_2, it will be designed for the same load but with a higher strength or a lower exclusion limit, say of 2 %. If the design of component R_2 was not made to achieve a planned strength coordination, its strength would have been (10 %) R_2. Figure A.6 illustrates the relations.

The ratio $\Phi_S = \dfrac{(10\ \%)R_1}{(10\ \%)R_2}$ is the factor related to the coordination of strength.

In the above example, $\Phi_S = \dfrac{(2\,\%)R_2}{(10\,\%)R_2} = \dfrac{(1-2{,}05v_{R2})}{(1-1{,}28v_{R2})}$ = 0,90

This result means that if the 10 % exclusion limit is used for the design of the weakest component, the strength of the next weakest component should be higher by a factor 1/0,91 = 1,10. The results depend essentially on the standard deviation of the next weakest component. Where v_R = 0,25, as is the case for foundations, Φ_S = 0,72 would result.

It can be easily shown that this factor remains fairly constant for different members N of components in series. In this case, Φ_s changes to:

$$\Phi_S = \frac{(2\,\%)\min_N R_2}{(10\,\%)\min_N R_2} \tag{A.15}$$

Using the above data we find that Φ_S varies from 0,91 for N = 1 to 0,94 for N = 100 (see Table B.5).

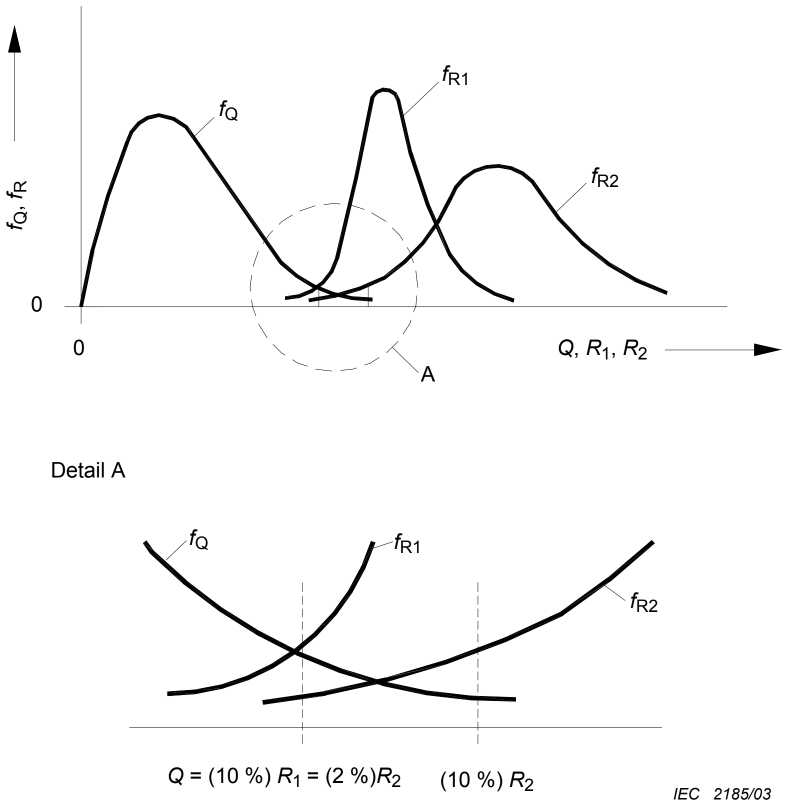


Figure A.6 – Coordination of strength by using different exclusion limits

A.1.3.4 Design for a target confidence level in the strength coordination

Because components of overhead lines have random strengths, it is impossible to guarantee with 100 % confidence that components will fail in a preferred sequence. A probability level lower than 100 % that the strength of one component will exceed the strength of another one has to be accepted.

The strength coordination factor Φ_S depends on the target probability of achieving the assumed sequence of failure. The following method can be used to derive Φ_S factors.

The target probability that the strength of component 2 exceeds the strength of component 1 has to be established, e.g. probability for $R_2 > R_1$ = 0,90 or

$$P\ (R_2 - R_1) > 0 = 0{,}90 = P(\text{sof}), \qquad \text{(A.16)}$$

where

R_1, R_2 is the strength of the components;

P(sof) is the probability of sequence of failure or coordination of strength.

Using statistical methods the factor Φ_S can be derived for various combinations of v_{R1} and v_{R2}, (v_{R1}, v_{R2} coefficient of variation of strength).

The factor Φ_S is related to the 10 % exclusion limit of the strength of the components compared

$$\Phi_S = (10\ \%)R_1 \,/\, (10\ \%)R_2$$

It is assumed that the density function of the strength follows a normal distribution.

Then the confidence for achieving a target coordination of strength can be expressed by the reliability index β_{sof} which is the number of standard deviations for which the Gaussian distribution yields the established confidence:

$$F\,(\beta_{sof}) = P_\phi \tag{A.17}$$

From tables for the Gaussian distribution the reliability index for the confidence in the preferred sequence of failure (β_{sof}) can be obtained:

For $P_\Phi = 0{,}90$; $\beta_{sof} = 1{,}28$

$P_\Phi = 0{,}98$; $\beta_{sof} = 2{,}05$, etc.

According to Equation (A.10):

$$\beta_{sof} = \frac{\overline{R}_1 - \overline{R}_2}{\sqrt{\sigma_{R1}^2 + \sigma_{R2}^2}} \tag{A.18}$$

applies, where

σ_{R1} = $v_{R1} \times \overline{R}_1$ standard deviation

σ_{R2} = $v_{R2} \times \overline{R}_2$ standard deviation

$\overline{R}_1, \overline{R}_2$ = mean component strengths.

With the introduction of the central safety factor:

$$\alpha = \overline{R}_2 \,/\, \overline{R}_1 \tag{A.19}$$

Equation (A.18) yields to:

$$\beta_{sof} = \frac{1-\alpha}{\sqrt{\alpha^2 \times v_{R_1}^2 + v_{R_2}^2}} \tag{A.20}$$

From Equation (A.20) the central safety factor α can be obtained by solving the quadratic equation:

$$\alpha^2\,[1 - (\beta_{sof} v_{R2})^2] - 2\alpha + 1 - (\beta_{sof} v_{R1})^2 = 0 \tag{A.21}$$

Table A.4 contains the results for $\beta_{sof} = 1{,}28$ and various coefficients of variation v_{R1} and v_{R2}.

Furthermore, since

$$(10\ \%)\ R_1 = (\overline{R}_1) \times (1 - 1{,}28\ v_{R1})$$

$$(10\ \%)\ R_2 = (\overline{R}_2) \times (1 - 1{,}28\ v_{R2})$$

$$\Phi_s = \frac{10\,\%R_1}{10\,\%R_2} = \frac{\overline{R}_1(1-1{,}28v_{R1})}{\overline{R}_2(1-1{,}28v_{R2})} = \frac{1\times(1-1{,}28v_{R1})}{\alpha\times(1-1{,}28v_{R2})} \quad \text{(A.22)}$$

The strength coordination factor Φ_S can be determined from Equation (A.22) after α has been calculated from Equation (A.21).

In Table A.4 values of α and Φ_S are given that ensure that component R_2 will fail after component R_1 with 90 % probability (P_Φ = 0,90).

Table A.4 – Values of central safety factor α and strength coordination factor Φ_S required to insure that component R_2 will fail after component R_1 with a 90 % probability

v_{R1} / v_{R2}	0,05		0,075		0,10		0,20	
	α	Φ_S	α	Φ_S	α	Φ_S	α	Φ_S
0,05	1,10	0,91	1,12	0,86	1,15	0,81	1,26	0,63
0,10	1,16	0,92	1,18	0,88	1,20	0,83	1,30	0,65
0,20	1,36	0,93	1,36	0,89	1,37	0,85	1,45	0,69
0,30	1,63	0,93	1,64	0,90	1,64	0,86	1,70	0,71
0,40	2,07	0,93	2,07	0,90	2,07	0,86	2,11	0,72

Strength coordination would be difficult and not cost efficient to choose a component with a large strength variation as the first component to fail. For example, as seen from Table A.4, when v_{R1} = 0,20, the characteristic strength of the next strongest components would have to be selected such that it would meet the limit loads when multiplied by about 0,7.

From Table A.4, it can be concluded that if suspension supports (usually v_R = 0,05 to 0,10) are designed as the weakest components, the characteristic strength of foundations (v_R is usually from 0,10 to 0,30) has to be multiplied by a factor of 0,83 to 0,93. In this case, there is 90 % confidence that foundations will not fail before the supported tower.

A.1.3.5 Number of components subjected to maximum load intensity

When the maximum intensity of a load event Q_T affects a large number of components, failure will be triggered by the weakest link (or component). This effect has to be considered when establishing the strength distribution or the effective exclusion limit. Assuming that the strength of components is not correlated, the strength distribution of a series of N components becomes $\min_N R$, where N is the number of components subjected to the maximum load intensity.

The density function of $\min_N R$ can be derived by analytical methods or simulation techniques, e.g. Monte-Carlo. However, since the 10 % exclusion limit is used as a reference in this standard (based on Equation (A.12) the exclusion limit e_N of N components can be obtained through the following relation:

$$e_N = 1 - [1 - e_1]^N \quad \text{(A.23)}$$

The exclusion probability of a system consisting of N components or elements can be calculated from the exclusion limit of the individual elements using this relation.

For design purposes according to this standard it has to be assumed that the exclusion limit e_N is 10 %. Therefore, the exclusion limit for each individual element must be chosen such that the requirement of e_N = 0,10 will met:

$$e_1 = 1 - (1 - e_N)^{1/N} \tag{A.24}$$

According to Equation (A.4), it follows that:

$$R(e) = \overline{R}\ (1 - u_e \times v_R)$$

The value u_e can be obtained from the standardized normal distribution (see C.2.1)

$$F_R\ (u_e) = e \tag{A.25}$$

and corresponds to the number of standard deviations for which Equation (A.25) is satisfied. In determining the strength of a component, the number of components subjected to the same load is considered by the strength factor Φ_N

$$\Phi_N = \frac{1 - u_{e1} \times v_R}{1 - u_{eN} \times v_R} \tag{A.26}$$

For example, let us calculate Φ_N for N = 10 and v_R = 20 %. Here, e_N is 0,10.

From Equation (A.24) $e_1 = 1 - (1\text{-}0{,}1)^{0{,}1}$ = 0,0105 results. From tables of standardized normal distribution it can be obtained

$$u_{eN} = 1{,}28 \text{ from } F_R(u_{eN}) = 0{,}10$$

and

$$u_{e1} = 2{,}31 \text{ from } F_R(u_{e1}) = 0{,}0105$$

With these data the strength factor Φ_N can be determined from Equation (A.26):

$$\Phi_N = \frac{1 - 2{,}31\, x\, 0{,}2}{1 - 1{,}28\, x\, 0{,}2} = 0{,}72$$

The significance of this result is important. When the maximum intensity of a load acts on 10 components the strength of which is represented by a normal density function with a coefficient of variation of 20 %, the reliability is lower than if this load would act just on one component. In order to obtain the same reliability in both cases, the nominal strength in the case of 10 components has to be chosen such that if multiplied by the strength factor Φ_N = 0,72 it will be able to withstand the corresponding design action. When v_R = 0,075 and 0,05, this factor would be 0,92 and 0,94, respectively.

The same consideration applies to insulator strings. The mechanical rating of an insulator string depends on the number of insulators in the string and on strength dispersion, v_R, of insulator units. Assuming v_R = 0,05, a string of 80 insulators each rated for R_c, has to be multiplied by 0,9, while a string of 10 of the same insulators has to be multiplied by 0,94. If v_R = 0,15, then the strength factor Φ_N becomes 0,68 and 0,81 respectively, thus underlining the importance of N when strength variation is high.

For many line components the log-normal distribution describes the variation of strength more precisely than the normal distribution, especially at the lower tail of the distribution. Therefore the strength factor Φ_N is determined for this type of distribution as well. Under this assumption Φ_N is

$$\Phi_N = \frac{R_{e1}}{R_{eN}} \tag{A.27}$$

where R_{e1} is the relative strength of each individual component of N components in series and R_{eN} is the target strength of all components in series. According to the example above R_{e1} is associated with an exclusion limit of 0,0105 and R_{eN} with that of 0,1. For the log-normal distribution the Gaussian distribution can be used

$$F_R\ (u) = e$$

where, according to Equation (C.12):

$$u = [\ln\ (R - p_1) - p_3]\ /\ p_2 \tag{A.28}$$

The term p_1 is zero.

The expressions p_2 and p_3 are given by Equations (C.17) and (C.18).

$$p_2{}^2 = \ln\ (v_R{}^2 + 1) \tag{A.28}$$

and

$$p_3 = \ln\ \overline{R} - \frac{1}{2}\ln\left(v_R^2 + 1\right) \tag{A.29}$$

For the above example which is carried out relatively to a mean strength, $\overline{R}$ is 1,0 and v_R = 0,20.

Hence

$p_2{}^2 = \ln\ (0{,}2^2 + 1) = 0{,}0392$; $p_2 = 0{,}198$;

$p_3 = -\ ½ \ln\ (0{,}2^2 + 1) = -0{,}0196$

with $F_R\ (u_{eN}) = 0{,}10$, $u_{eN} = -1{,}28$ results from the Gaussian distribution, and $u_{e1} = -2{,}31$ for $F_R\ (u_{e1}) = 0{,}0105$ as in the case of the normal distribution. From Equation (A.28) it is received

$$\ln R_{eN} = u_{eN}\ p_2 + p_3$$

and

$$\ln R_{1N} = u_{R1}\ p_2 + p_3$$

Therefore

$$\ln R_{eN} = -\ 1{,}28 \times 0{,}198 - 0{,}0196 = -\ 0{,}273;\ R_{eN} = 0{,}761;$$

and

$$\ln R_1 = -\ 2{,}31 \times 0{,}198 - 0{,}0196 = -\ 0{,}477;\ R_{1N} = 0{,}621.$$

From Equation (A.27) it follows Φ_N = 0,621 / 0,761 = 0,82 instead of 0,72 which was obtained from the Gaussian distribution.

Table A.5 depicts strength factors Φ_N depending on the number of components *N* in series or in parallel subjected simultaneously to the critical load obtained under the assumption of a normal and a log-normal distribution of their strength. The values for the latter are given in parenthesis. Values derived from other distributions can be used if more representative of the component being designed.

Table A.5 – Strength factor Φ_N related to *N* components in series subjected to the critical load

N	Strength coefficient of variation v_R						
	0,05	**0,075**	**0,10**	**0,15**	**0,20**	**0,25**	**0,30**
1	1,00	1,00	1,00	1,00	1,00	1,00	1,00
2	0,98	0,98 (0,97)	0,97 (0,97)	0,94 (0,95)	0,91 (0,93)	0,87 (0,92)	0,84 (0,90)
5	0,96	0,94 (0,94)	0,92 (0,93)	0,85 (0,89)	0,80 (0,86)	*0,72 (0,83)*	*0,64 (0,80)*
10	0,94 (0,95)	0,92 (0,93)	0,89 (0,90)	0,81 (0,86)	*0,72 (0,82)*	*0,62 (0,77)*	*0,51 (0,73)*
20	0,93 (0,94)	0,90 (0,91)	0,85 (0,88)	0,77 (0,83)	*0,66 (0,77)*	*0,53 (0,73)*	*0,38 (0,68)*
40	0,92 (0,93)	0,87 (0,89)	0,83 (0,86)	*0,72 (0,80)*	*0,59 (0,74)*	*0,44 (0,69)*	*0,26 (0,64)*
60	0,91 (0,92)	0,86 (0,88)	0,81 (0,85)	*0,70 (0,78)*	*0,56 (0,72)*	*0,40 (0,67)*	*0,20 (0,62)*
80	0,91 (0,92)	0,86 (0,88)	0,80 (0,84)	*0,68 (0,77)*	*0,53 (0,71)*	*0,36 (0,65)*	*0,16 (0,60)*
160	0,90 (0,91)	0,85 (0,87)	0,79 (0,83)	*0,67 (0,76)*	*0,52 (0,69)*	*0,34 (0,62)*	*0,13 (0,57)*
NOTE Values in parenthesis refer to log-normal distribution.							

For high values of v_R and *N* the value Φ_N is very sensitive to the choice of the distribution function. In these cases, the normal distribution function will not be adequate because of its lower tail that can extend to negative values of *R*. Therefore, care and engineering judgement is necessary in the selection of an appropriate distribution functions. Since these considerations refer to the lower tail of the distribution, the log-normal distribution seems more adequate than the normal distribution. The range of values where the difference in Φ_N is more than 10 % is separated in Table A.5 and shown in italic characters.

A.2 Strength of line components

A.2.1 Calculation of characteristic strength

The characteristic strength is defined as the strength guaranteed with a given probability.

If $\overline{R}$ is the mean strength of a component and v_R its coefficient of variation, then the

characteristic strength R_c is given by equation:

$$R_c = \overline{R}\ (1 - u_e\ v_R) \qquad \text{(A.30)}$$

The value of v_R depends on the type of material and the fabrication practice (quality control). The variable factor u_e depends on the distribution function of the strength of the component and on the probability of exceeding the guaranteed strength, represented by the exclusion limit *e*.

The characteristic strength of line components in most countries corresponds to an exclusion limit (probability of not being achieved) lower than 10 % and usually ino the order of 2 % to 5 %. Assuming a characteristic strength with a higher exclusion limit would produce a significant number of under-strength components and a very low exclusion limit may not be cost-effective, specially for components with high v_R. Thus, values from 2 % to 5 % correspond to a practical economic balance. If a normal distribution is assumed for strength R, u_e would thus vary between 1,60 and 2,10.

For example, in order to guarantee a minimum yield point of 300 MPa for a given grade of steel, a manufacturer, knowing that the coefficient of variation is 0,05, will generally produce a steel which has an mean strength of 300 / (1 – 2,10 × 0,05) = 340 MPa. The probability of not meeting the minimum strength (or the characteristic strength) is quite low and is in the order of 2 %. The same approach applies to insulators where it was found from compiled strength data that the characteristic strength corresponds to a very low exclusion limit (approximately 0,1 %).

Consequently, the exclusion limit of 10 % used in the reliability Equation (A.6) can be related to the characteristic value by means of:

$$(10\%)R = \left(1 - 1{,}28 v_R\right)\overline{R} = \frac{\left(1 - 1{,}28 v_R\right)R_c}{1 - u_e v_R} \qquad \text{(A.31)}$$

or

$$(10\ \%)R = \Phi_c\ R_c \qquad \text{(A.32)}$$

If the value of u_e is not known, it can be estimated according to Table A.6 which is based on the frequency of rejects calculated from the normal distribution.

Table A.6 – Values of u_e associated to exclusion limits

	Estimated frequency of rejects		
	Frequent	**Some**	**Rare**
Exclusion limit *e*	About 10 %	2 % to 5 %	< 2 %
u_e	1,28	1,6	2,1

Φ_c is a correction factor that can be applied to the characteristic strength R_c if there is enough evidence or data to warrant that the exclusion limit of R_c is different from 10 %.

$$\Phi_c = (1 - 1{,}28\ v_R) / (1 - u_e\ v_R) \qquad \text{(A.33)}$$

In typical cases Φ_c can be considered equal to 1,0 which should normally lead to a satisfying design reliability.

A.3 Temperature measurements and their interpretation

A.3.1 General

The need for temperature data is, in this standard, related to design minimum temperature and atmospheric icing.

Temperature variations in the conductor are due to either convection of colder/warmer air combined with wind, or changes in the radiation balance due to sunshine, cloud cover, etc. mostly in still weather. Radiation from surrounding buildings, vegetation, etc. may influence thermometers as well. For these reasons it is important that thermometers are properly shielded and ventilated, and that the location is appropriate for the purpose of measurements.

Radiation shields are, for standard meteorological measurements, mostly made of wood. Smaller screens made of metal or plastic have less thermal mass and take more rapid fluctuations in temperature. Averaging periods of 1 min to 3 min are recommended for electronically controlled measurements in such screens.

The standard measuring height is 2 m above ground.

The required accuracy of data may vary with the purpose of the measurements. For icing studies, especially wet snow, sufficient accuracy must be considered.

As a general rule, it is suggested to consult meteorological institutions in order to optimize the location(s), selection of sensors, data acquisition and interpretation of any weather measurements. Temperature recordings, like any other meteorological measurements should be linked to standard meteorological measurements from, which are called here, reference stations. This will reduce the especially the length of necessary measurements, but also existing statistics may relatively easily be transferred to the site in question.

A.3.2 Location of reference measurements

Official meteorological observations are regularly taken from open, flat terrain with few trees and scattered buildings within a radius of some kilometres, typically airports. Data from such stations are therefore mostly representative for wide areas, and are relatively easy to correlate with other sites.

A.3.3 Localization of thermometers

General rules for the localization of thermometers are not possible to specify, however the measurement sites should be evaluated regarding:

– vegetation;
– forests;
– buildings;
– ventilation;
– cold air flows or basins (in winter);
– heating of (sloping) ground by sunshine (in summer);
– radiation from surrounding buildings, forests, rock, etc.;
– height above ground (standard height 2 m).

The importance of these effects should be related to the required accuracy for the study.

When the temperature measurements are performed in an icing environment, it is important to monitor the icing on the screen itself. Accumulated ice on the screen will generally increase the thermal mass and hence the time constant of the sensing system in sub-freezing temperatures. When the temperature of the surrounding air rises above the freezing point, the ice starts to melt and the temperature within the screen remains at 0 °C until the ice has melted away. The only way to ensure good temperature data under icing conditions is careful monitoring and cleaning of the screen.

For special stands higher than about 25 m it is recommended to measure the temperature at least at two levels.

A.3.4 Interpretation of the measurements

When the sites for temperature measurements are carefully selected and the sensors properly installed and maintained, the correlation of data is, in most cases, straightforward. In most cases, the statistical treatment of data, such as extreme value analyses, should be carried out on the data from the reference station. Values from the reference station may then be transferred to the local site by means of correction factors found from the parallel measurements.

The data analysis may often be restricted to the weather conditions in question, for instance to situations with probability of icing.

If the purpose is only to establish the minimum temperature for conductor tension ($T_{min\ av}$), it is sufficient to find the correlation between the measuring site and the reference station during cold spells.

If the minimal temperatures follow an extreme value distribution law, Gumbel type I of all formulations developed in Clause C.4 can be applied. The daily minimum temperatures should be recorded. The mean daily minimum temperatures will be deduced, as well as the lowest annual minimum temperature T_{min} recorded over a certain number or years. These values will be used to compute the mean annual minimum temperature ($T_{min\ av}$) as well as the coefficients C and β of the following relation:

$$\frac{T_R}{T_{\min av}} = 1 + C - \beta \cdot \ln [-\ln (1 - P_T)] \qquad (A.34)$$

where T_R is the value of the lowest annual minimum temperature having a probability P_T of being exceeded once a year.

A.3.5 Duration of measurement

Temperature measurements for design and planning purposes should last for a period long enough to establish correlation with the reference station(s) with the required accuracy, 1 to 2 years (seasons) are in most cases sufficient.

A.3.6 Application to other sites

When data from one site is to be applied on other sites it is worth noting, as mentioned in A.3.2, that the air temperature may vary less horizontally than vertically. Therefore, the temperature measured at one site may be applicable to a rather wide area with similar terrain in the same level.

A.4 Determination of the meteorological reference wind speed

A.4.1 Roughness of terrain

Wind action is influenced by the terrain roughness. The greater this roughness, the more turbulent and slower is the wind. The terrain roughness has an influence both on the determination of the wind speed for the design and on the determination of the gust factor.

Four categories of terrain, of increasing roughness, are considered as indicated in Table A.7.

Table A.7 – Definition of terrain category

Terrain category	Characteristic of the terrain crossed by a line
A	Large stretch of water up-wind, flat coastal area, flat dessert
B	Open country with very few obstacles, for instance moorlands or cultivated field with a few trees or buildings
C	Terrain with numerous small obstacles of low height (hedges, trees and buildings)
D	Suburban areas or terrain with many tall trees

Lines crossing highly urbanized areas should be considered in a D terrain roughness. The value of the terrain roughness is very difficult to assess for these areas. However, due to the derived higher reliability of lines in these areas, design according to terrain category B or C are proposed.

For a line that follows the ridge of a hill, a terrain roughness which is smoother by one category than the one chosen for the area should be selected in order to be conservative. For a line running along a valley, the C roughness should be chosen for all cases, whatever the terrain characteristics may be.

A.4.2 Assessment of meteorological measurements

Wind action is evaluated on the basis of the reference wind speed V_R defined as mean value of the wind during a 10 min period at a level of 10 m above ground (see Table A.7).

Usually, meteorological stations (except those along the coast or in urban areas) are placed in areas of B terrain category. The reference wind speed in terrain category B is V_{RB}.

Nevertheless, the meteorological wind speed may be recorded in a terrain category x site at 10 m above the ground as a mean value over a period of time t in s. Let $V_{x,t}$ be this speed. (If it is not measured at 10 m height above ground, the data should be adjusted first to this level.)

The curves of Figure A.7 enable to determine the ratio $V_{x,t}$ / $V_{x,10\ min}$ as a function of the averaging period for each category of roughness at the location of the meteorological site. These values may be used in the absence of local data or studies as indicated in the note in 6.2.3.

For a known $V_{x,10\ min}$, V is given by the following relation:

$$V = V_{x,10\ min} / K_R \qquad (A.35)$$

Values for K_R are given in Table A.8.

Table A.8 – Factors describing wind action depending on terrain category

Factor	Terrain category			
	A	B	C	D
Z_0 (roughness length) (m)	0,01	0,05	0,30	1,00
α	0,10 to 0,12	0,16	0,22	0,28
K_R	1,08	1,00	0,85	0,67

The variation of *V* in terms of height was not taken into account, as anemometers are, most of the time, placed at a height of about 10 m above surrounding ground. If this height *z* (m) differs from 10 m, the variation of wind speed with height *z* can be derived from the so-called "power law", shown in (A.36) may be used. α is found in Table A.8.

$$V_z = V_R \left(\frac{z}{10}\right)^{\alpha} \tag{A.36}$$

Or more generally:

$$V_{z1} = V_{z2} \left(\frac{z1}{z2}\right)^{\alpha} \tag{A.37}$$

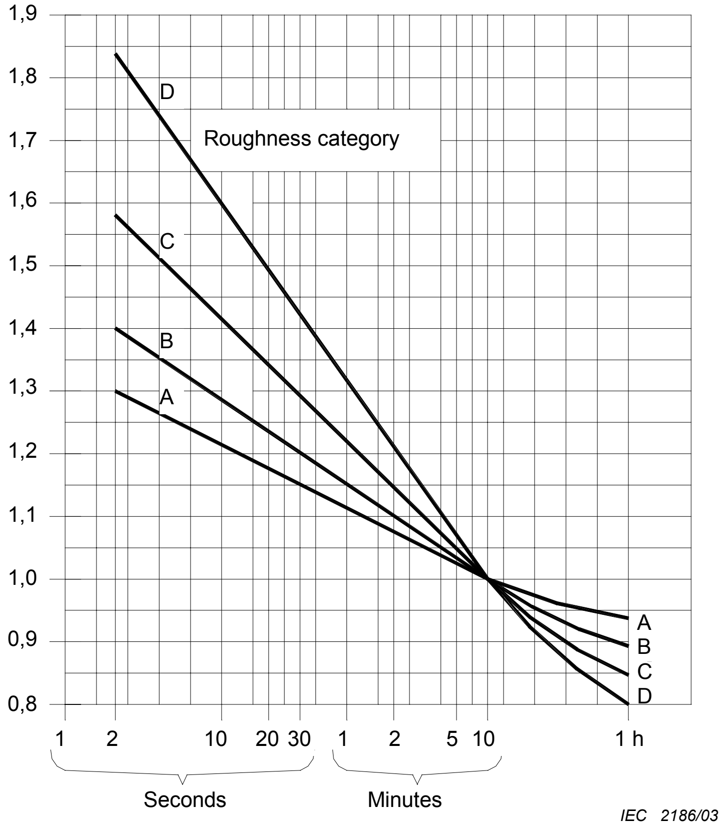


Figure A.7 – Relationship between meteorological wind velocities at a height of 10 m depending on terrain category and on averaging period

A.4.3 Determination from gradient wind velocities

Where meteorological stations are remote from the locations considered for the erection of the line, the gradient wind speed, defined as the speed at the level on the top of the earth's boundary layer, which is 800 mm to 1 000 m above ground, may be used as a basis for assessment of design wind velocities.

The gradient wind action is characterized by the mean value of yearly maximum gradient wind velocities $\overline{V}_G$ and its standard deviation σ_{VG}. From the wind speed $\overline{V}_G$ the mean of the yearly maxima $\overline{V}_m$ (10 m above ground) can be approximated by the following equation:

$$\overline{V}_m \text{ (B)} = 0{,}5\ \overline{V}_G \tag{A.38}$$

Data for $\overline{V}_G$ can usually be obtained from national weather services.

A.4.4 Wind measurements

Wind measurements should be handled according to guidelines given by the World Meteorological Organization (WMO), especially in order to compare collected data with longer time series of standardized measurements. The following parameters should be considered:

– open and representative location for the purpose (type of terrain);
– height above ground (standard 10 m);
– averaging time (standard 10 min for mean wind speed, 3 s to 5 s for gusts);
– wind direction.

It is recommended that questions regarding instrumentation, selection of measuring sites, logging protocols, data analyses etc. are discussed with experienced experts in wind engineering.

For the purpose of this standard, it is assumed that the yearly maximum wind velocities are recorded over a period of *n* years and can be processed as described in B.2.1.

A.4.5 Wind action

A.4.5.1 General

A procedure is described in A.4.2 to compute *V* whenever the meteorological speed is not measured in the standard conditions.

The yearly maximum wind speed V_m is the maximum of *V* measured over a year.

The loading assumptions should consider

– high wind assumption,
– reduced wind associated with minimum temperature.

Minimum temperature assumption is not usually critical for suspension supports, but should be checked for tension or dead-end supports, particularly for short spans.

A.4.5.2 Reference wind speed for design

a) Reference wind speed

The determination of the reference wind speed V_R depends upon the reliability level for which the line will be designed.

The reference wind speed V_R is determined from the mean speed using Equations (B.7) or (B.8) of Annex B from the mean speed $\overline{V}_m$ of the yearly maximum velocities V_m and the standard deviation σ_{Vm} of the statistical distribution of these velocities as well as from the return period T and the parameters C_1 and C_2, which depend on the numbers of years with observations.

Table A.9 – Values of reference wind speed V_R

Return period T (a)	V_R / $\overline{V}_m$		
	v_v = 0,12	v_v = 0,16	v_v = 0,20
50	1,31	1,41	1,52
150	1,41	1,55	1,69
500	1,53	1,70	1,88

In Table A.9 the ratio V_R / $\overline{V}_m$ is given for typical coefficients of variations. In Europe, a value of v_v = 0,12 was found in several countries. The values in this table are given for cases where the number of years of observations is very large. In other cases, refer to Clause B.2 for deriving the ratio of V_R / $\overline{V}_m$ considering the number of years with observations.

For areas subject to very high wind speed but infrequent winds (such as typhoons, downbursts, or tornadoes), a special study is required for the determination of the distribution law of maximum values, which cannot be deducted solely from the yearly maximum velocities.

If the transmission line is to be designed in a terrain type different from B category, the reference wind speed V_R shall be multiplied by terrain factor V_R. This factor is integrated in the calculation of dynamic wind pressure of A.4.5.3 where the new wind speed becomes:

Reference wind speed for computing wind forces in a terrain type other than B category

$$V_{RX} = K_R \, V_{RB} \qquad \text{(A.39)}$$

where K_R is a factor which takes into account the roughness of the ground at the location of the line and in the surrounding area (see Table A.8).

For sites of intermediate roughness, K_R can be interpolated. In estimating the value of the terrain roughness, it is necessary to consider the foreseeable changes in the surrounding of the route of the line. When deriving wind actions just from a single series of measurements, the consistency of the results has to be cross-checked against experience with comparable lines and terrain.

The wind velocities defined above for computation shall be considered as occurring at an air temperature equal to the mean of the daily minimum temperatures, peculiar to this site. The mean daily minimum temperature may be obtained by means of analysis of the recordings over a certain number of years in a meteorological station as close as possible to the location of the line. As an alternative, it is admissible to take as a coincident air temperature the minimum temperature defined hereinafter increased by 15 °C.

b) Reduced wind

A reduced wind speed should be combined with minimum temperature. The minimum temperature should be considered as being equal to the yearly minimum value, having a probability of occurrence of 2 % or a return period of 50 years. A method for the determination of this value is given in A.3.4. When the line is located in an area where the minimum air temperature can be influenced by the local topography, it is necessary to take this influence into account.

The reduced wind speed will be equal to the reference wind speed V_R chosen for the high wind assumption multiplied by a coefficient chosen according to local meteorological conditions. Where there is no reliable knowledge of local conditions, a value of 0,6 for this coefficient is suggested.

A.4.5.3 Unit-action of the wind on any element of the line and dynamic reference pressure

The characteristic value *a* of the unit-action in N/m^2, due to the wind acting horizontally, perpendicularly to any element of the line (conductors, insulators, supports or parts of them) is given by the following expression:

$$a = q_0 \, C_x \, G \qquad \text{(A.40)}$$

where

- q_0 is the reference dynamic wind pressure;
- C_x is the drag coefficient depending on the shape of the element being considered;
- G is the combined wind factor which takes into account the turbulence of the wind. It varies in terms of the dynamic response of the element being considered (gust response). It also depends on the height of this element above the ground and, for conductors, on the length of the span, as given in 6.2.6.

The dynamic reference pressure q_0 is given in terms of the reference wind speed V_{RX} at 10 m above ground at the location of the line (see Equation (A.39)).

$$q_0 = \frac{1}{2}\mu \times \tau \times V_{RB}^2 K_R^2 = \frac{1}{2}\mu \times \tau \times V_{Rx}^2 \qquad \text{(A.41)}$$

where

- μ is the air mass per unit volume (equal to 1,225 kg/m³ at a temperature of 15 °C and under a normal atmospheric pressure of 1 013 mbar);
- τ is the correction factor depending on air temperature and altitude (see Table 5);
- q_0 is expressed in N/m^2 and V_{RX} in m/s.

A.4.6 Wind load on conductors (general case)

In 6.2.6 detailed information is given on wind loads on individual line components. This subclause describes wind loads on conductors in a general case of an angle tower and a given wind direction.

The wind load A_c due to the effect of the wind upon a span length L, applied at each attachment point of this span and perpendicularly to the span, is given by Equation (8) (see also Figure A.8):

$$A_c = q_0\, C_{xc}\, G_c\, G_L\, d\, \frac{L_i}{2} \sin^2 \Omega \tag{A.42}$$

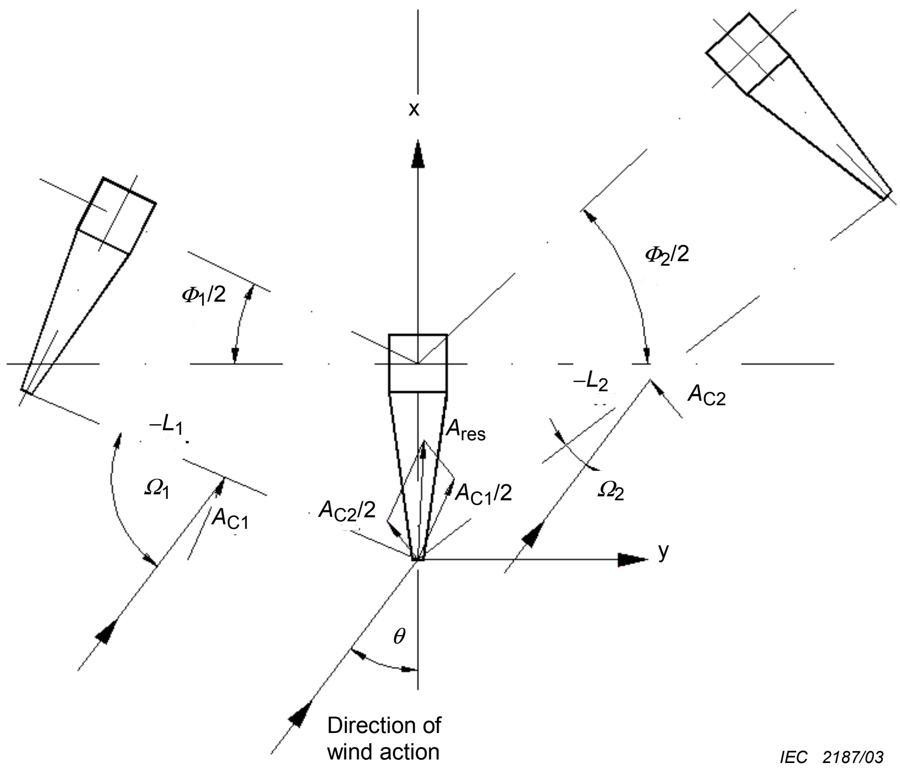


Figure A.8 – Wind action on conductors and resultant wind load on support

From Equation (A.42) the wind action on a support can be determined from Figure A.8.

If the wind direction is determined by the angle θ between wind direction and support crossarm axis and Φ_1 and Φ_2 are the compliments to the line angles it follows

$$\Omega_1 = 90° - (\theta - \Phi_1/2) \text{ and } \Omega_2 = 90° - (\theta + \Phi_2/2) \tag{A.43}$$

For the support according to Figure A.8, the wind load in direction of the crossarm is:

$$A_{cx} = (A_{c1} / 2) \times \cos(\Phi_1 / 2) + (A_{c2} / 2) \times \cos(\Phi_2 / 2) \tag{A.44}$$

and perpendicularly to this direction

$$A_{cy} = (A_{c1} / 2) \times \sin(\Phi_1 / 2) - (A_{c2} / 2) \times \sin(\Phi_2 / 2) \tag{A.45}$$

Combining (A.42), (A.43), (A.44) and (A.45) yields the wind load in direction of the crossarm

$$A_{cx} = q_0\ C_{xc}\ G_c \times d \left[G_{L1} \frac{L_1}{2} \sin^2\left(90° - (\theta - \phi_1 / 2)\right) \cos\left(\frac{\phi_1}{2}\right) + G_{L2} \frac{L_2}{2} \sin^2\left(90° - (\theta + \phi_2 / 2)\right) \cos\left(\frac{\phi_2}{2}\right) \right] \tag{A.46}$$

and perpendicularly

$$A_{cy} = q_0\ C_{xc}\ G_c \times d \left[G_{L1} \frac{L_1}{2} \sin^2\left(90° - (\theta - \phi_1 / 2)\right) \sin\left(\frac{\phi_1}{2}\right) + G_{L2} \frac{L_2}{2} \sin^2\left(90° - (\theta + \phi_2 / 2)\right) \sin\left(\frac{\phi_2}{2}\right) \right] \tag{A.47}$$

For suspension supports without a line angle $\Phi_1 = \Phi_2 = 0$ applies and from (A.46) it follows

$$A_{cx} = q_0 \times C_{xc} \times G_c \times G_L \times d \times \frac{L_1 + L_2}{2} \sin^2 (90 - \theta) \tag{A.48}$$

and in case of wind perpendicular to line direction

$$A_{cx} = q_0 \times C_{xc} \times G_c \times G_L \times d \times \frac{L_1 + L_2}{2} \tag{A.49}$$

From (A.47) $A_{cy} = 0$ results in these cases.

In case of wind action in direction of the crossarm axis of an angle support, erected in the bisector of the line angle Φ, $\theta = 0$ applies and with $\Phi_1 = \Phi_2 = \Phi$ it follows from (A.46)

$$A_{cx} = q_0 \times C_{xc} \times G_c \times G_L \times d \times \frac{L_1 + L_2}{2} \cos^3\left(\frac{\phi}{2}\right) \tag{A.50}$$

and from (A.47)

$$A_{cy} = q_0 \times C_{xc} \times G_c \times G_L \times d \times \frac{L_1 - L_2}{2} \cos^2\left(\frac{\phi}{2}\right) \times \sin\left(\frac{\phi}{2}\right) \tag{A.51}$$

Only if $L_1 = L_2$, then $A_{cy} = 0$.

In Equations (A.48 to A.51) G_L should be determined for $L = (L_1 + L_2) / 2$.

A.4.7 Equations of curves

NOTE 1 During circulation of the first committee draft (CD), a request was made to provide equations of curves appearing in this standard that could be used during overhead line design by computers. The equations provided hereafter describe the curves in various figures of the standard. These equations are based on proposals made by one national committee.

NOTE 2 Many of the curves in this standard were originally developed in previous IEC/TC11 documents based on a combination of theoretical studies and experimental results that were fine tuned according to experience.

A.4.7.1 Equations for G_c – Figure 3

$G_c = 0{,}2914 \times \mathrm{Ln}(x) + 1{,}0468$ (terrain type A)

$G_c = 0{,}3733 \times \mathrm{Ln}(x) + 0{,}9762$ (terrain type B)

$G_c = 0{,}4936 \times \mathrm{Ln}(h) + 0{,}9124$ (terrain type C)

$G_c = 0{,}6153 \times \mathrm{Ln}(h) + 0{,}8144$ (terrain type D)

A.4.7.2 Equation for G_L – Figure 4

$G_L = 4 \times 10^{-10} \times L^3 - 5 \times 10^{-7} \times L^2 - 10^{-4} \times L + 1{,}0403$

A.4.7.3 Equations for G_t – Figure 5

$G_t = -0{,}0002 \times h^2 + 0{,}0232 \times h + 1{,}4661$ (terrain type A)

$G_t = -0{,}0002 \times h^2 + 0{,}0274 \times h + 1{,}6820$ (terrain type B)

$G_t = -0{,}0002 \times h^2 + 0{,}0298 \times h + 2{,}2744$ (terrain type C)

$G_t = -0{,}0002 \times h^2 + 0{,}0384 \times h + 2{,}9284$ (terrain type D)

A.4.7.4 Equation for C_{xt} – Figure 7 (flat sided members)

$C_{xt1,2} = 4{,}1727 \times \chi^2 - 6{,}1681 \times \chi + 4{,}0088$

A.4.7.5 Equation for C_{xt} – Figure 8, (round sided members)

$C_{xt1,2} = 0{,}2293 \times \chi^3 + 2{,}7091 \times \chi^2 - 3{,}1323 \times \chi + 2{,}2002$

A.4.7.6 Equations for C_{xTc} – Figure 9

$C_{xTc} = 1{,}2$ when $Re < 3 \times 10^5$

$C_{xTc} = 0{,}75$ when $Re > 4{,}5 \times 10^5$

$C_{xTc} = -1{,}1098 \times \mathrm{Ln}(Re) + 15{,}197$, when $3 \times 10^5 < Re < 4{,}5 \times 10^5$

A.4.8 Wind effect on conductor tension

Wind acting on conductors will cause an increase in their mechanical tension that can be computed with standard sag-tension methods. Two cases of wind and temperature combinations should be checked as stated in A.4.5.2.

Where a long series of spans is separated by suspension insulators, the ruling span concept may be used for tension calculations. It is important to note that the ruling span concept implies that the same wind pressure applies to all spans between dead-end insulators. This is a conservative assumption if the number of suspension spans is large. In this case, it is acceptable to reduce the wind pressure calculated with Equations (A.41) and (A.42). A reduction factor of 0,6 to 1,0 applied to the wind pressure may be used. However, caution should be exerted when using this reduction factor because some supports may be used in sections with few spans between dead-ends.

A.4.9 Number supports subjected in wind action, effect of length of line

Gusts with maximum wind speed are limited in width. An individual gust will, therefore hit only one support and the adjacent spans. Nevertheless to take care of the several gusts with approximately the same magnitude it is proposed to assume that five supports are hit in flat or rolling terrain and two in mountains.

For long lines, the probability to be hit by extreme wind actions is higher than for short lines. The effect depends on many aspects, such as variation of terrain and climate, design of supports adjusted to the terrain and the loads to be expected there. The design of the line should aim at the same reliability of the total line related to the service life of the line.

Lines with relatively short length up to 100 km should be designed for a reliability level as proposed in subclauses A.1.2.4 and A.1.2.5. For longer lines, in order not to increase the probability of failure, the return periods of chosen design assumptions should be extended so as to achieve the overall reliability. The adjustment of return periods is not required if map of wind data has already been adjusted to take into account the space covered by service area.

A.5 Atmospheric icing

A.5.1 General

Atmospheric icing is a general term for a number of processes where water in various forms in the atmosphere freezes and adheres to objects exposed to the air. Generally, there are two types of icing which are named according to the main processes

– precipitation icing, and
– in-cloud icing.

A third process, where water vapour is transformed directly into the ice phase and forms so-called “hoar frost”, does not lead to significant loadings and is not considered further.

Precipitation icing occurs in several forms, among which the most important are

– freezing rain,
– wet snow accretion, and
– dry snow accretion.

A.5.2 Precipitation icing

A.5.2.1 Freezing rain

When raindrops or drizzle fall through a layer of cold air (sub-freezing temperatures), the water droplets become supercooled. Therefore, they are still in a liquid water phase and do not freeze before they hit the ground or any object in their path. The resulting accretion is a clear, solid ice called glaze, often with icicles. This accretion is very hard and strong, and therefore difficult to remove. The density is 800-900 kg/m^3, depending on the content of air bubbles.

Freezing rain occurs mostly on wide plains or basins where relatively deep layers of cold air accumulate during spells of cold weather. When a low pressure system with a warm front with rain penetrates the area, the cold (and heavier) air may remain near the ground and thus favour the formation of glaze (temperature inversion). Such a situation may persist until the upper winds have managed to mix the cold surface layer of air with the warmer air aloft.

A similar situation may occur in the overlapping zones of cold air and warm air systems. The warmer air, often with precipitation, is lifted over the colder air and forms a frontal zone where precipitation is enhanced.

Usually there are only moderate winds during freezing rain events. Hence the amount of accreted ice depends on the precipitation rate and duration.

A.5.2.2 Wet snow

Normally, the temperature increases as snow flakes fall through the atmosphere. If the air temperature near the ground is above freezing, the snow flakes start to melt when passing the 0 °C isotherm and the flakes contain a mixture of ice and water (at 0 °C) until they eventually melt totally into raindrops if the warm layer is deep enough. As long as they are only partly melted they will adhere to objects in the airflow.

The density may vary widely (100-800 kg/m^3), but mostly from about 400-600 kg/m^3. The density and intensity of accreted wet snow depends on the precipitation rate, wind speed and temperature. If the temperature drops below 0 °C after the accretion, the layer will freeze into a hard and dense layer with strong adhesion to the object.

Wet snow may also freeze on objects in colder air near the ground as in the case of freezing rain.

NOTE The density of wet snow accretion (type of precipitation icing) usually increases with wind speed, thus resulting in a smaller area exposed to wind pressure. In such case, it is possible that the resulting forces on the cables subjected to increased wind speed (height variation of Figure 11) may be less critical than at lower wind speed at 10 m reference height.

A.5.3 In-cloud icing

In-cloud icing is a process whereby supercooled water droplets in a cloud or fog, freeze immediately upon impact on objects in the air flow, e.g. overhead lines in mountains above the cloud base.

The ice growth is said to be dry when the transfer of potential transfer of heat away from the object is greater than the release of the latent heat of fusion. The resulting accreted ice is called soft or hard rime according to its density which is typically 300 kg/m^3 for soft rime and 700 kg/m^3 for hard rime.

The ice growth is said to be wet when the heat transfer rate is less than the rate of latent heat release. Then the growth takes place at the melting point, resulting in a water film on the surface. The accreted ice is called glaze with a density of approximately 900 kg/m^3.

The icing rate varies mainly as a result of the following:

– liquid water content of the air;
– median volume droplet size of the spectrum;
– wind speed;
– temperature;
– dimensions of the iced object.

At temperatures below –10 °C the water content of the air becomes smaller and less icing occurs. However, 8 kg/m was recorded in Switzerland with a temperature below –20 °C and strong winds.

Under the same conditions the ice accretion rate will be greater for a small object than for a large one. Thus, heavy ice loadings are relatively more important for conductors than solid supports.

It should be noted that the heaviest in-cloud icing for specific locations, e.g. coastal mountains is usually due to a combination of wet-snow and hard rime.

A.5.4 Physical properties of ice

The physical properties of atmospheric ice may vary within rather wide limits. Typical properties are listed in Table A.10.

Table A.10 – Physical properties of ice

Type of ice	Density kg/m³	Adhesion	Appearance		Cohesion
			Colour	Shape	
Glaze ice	700 – 900	Strong	Transparent	Cylindrical icicles	Strong
Wet snow	400 – 700	Medium	White	Cylindrical	Medium to strong
Hard rime	700 – 900	Strong	Opaque to transparent	Eccentric pennants into wind	Very strong
Soft rime	200 – 600	Medium	White	Eccentric pennants into wind	Low to medium

A.5.5 Meteorological parameters controlling ice accretion

Table A.11 gives typical values of parameters that control the ice accretion.

Table A.11 – Meteorological parameters controlling ice accretion

Type of ice	Air temperature t °C	Mean wind speed V m/s	Droplet size	Liquid water content	Typical storm duration
Glaze ice	$-10 < t < 0$	Any	Large	Medium	Hours
Wet snow	$0 < t < 3$	Any	Flakes	Very high	Hours
Hard rime	$-10 < t < 1$	$10 < V$	Medium	Medium to high	Days
Soft rime	$-20 < t < 1$	$V < 10$	Small	Low	Days

The transition between soft rime, hard rime and glaze for in-cloud icing is mainly a function of air temperature and wind speed as shown in Figure A.9. However, the curves in Figure A.9 shift to the right with increasing liquid water content and with decreasing object size.

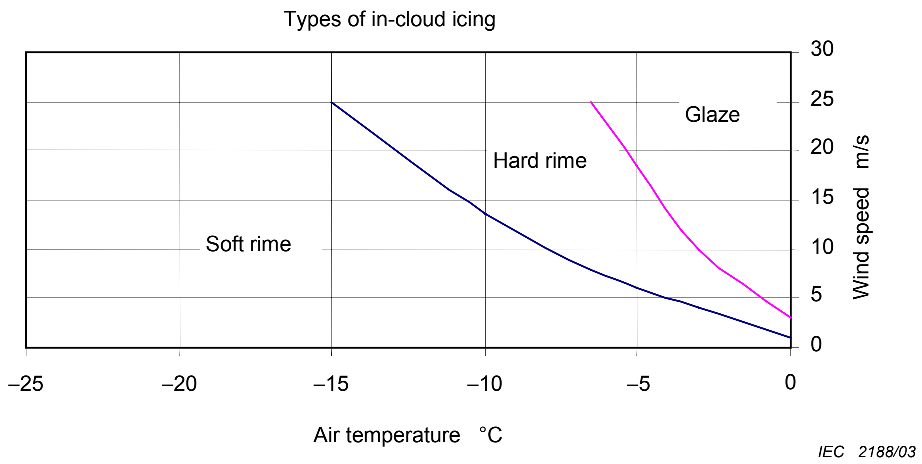


Figure A.9 – Type of accreted in-cloud icing as a function of wind speed and temperature

A.5.6 Terrain influences

A.5.6.1 In-cloud icing

The regional and local topography (large and medium scale) modifies the vertical motions of the atmosphere and hence the cloud structure and icing. Coastal mountains along the windward side of the continents act to force moist air upwards, leading to a cooling of the air with condensation of water vapour and droplet growth, eventually with precipitation. The most severe in-cloud icing occurs above the condensation level and the freezing level on freely exposed heights, where mountain valleys force moist air through passes and thus both lift the air and strengthen the wind.

On the leeward side of the mountains the descent of an air mass results in internal heating of the air and evaporation of droplets, eventually with a total dissolution of clouds. A local shelter of hills not more than 50 m higher on the windward side may give a significant reduction in ice loadings. For this reason, routes in high mountains may very well be suited for overhead lines, provided they are sheltered against icing wind directions.

A.5.6.2 Precipitation icing

In general, precipitation icing may occur at any altitude. However, the probability of precipitation icing is generally greater in the valley basin than half way up the valley sides because of higher occurrence of cold air. Both freezing rain and wet snow may occur on large plains.

The greatest amounts of wet snow may be formed where the transverse wind component is strongest. Hence, an overhead line along a valley has fewer accretions than a line crossing the valley.

However, smooth hills or mountains transverse to the wind may cause the wind to strengthen on the leeward side, especially if there are no obstacles to such a flow on this side. Combined with wet snow, such hillsides may have significant failure probabilities for high ice loads combined with high wind velocities.

A.5.7 Guidelines for the implementation of an ice observation program

At the current time of writing, there seems to be practically no indirect way of getting proper data for design, although significant efforts have been made to develop models based on meteorological data and the collection of general experience from the areas of interest. As for any other type of structure depending on extreme values of wind speed, snow depth or temperature, the transmission line designer needs data and measurements of the most critical design parameters. Therefore, a program for collecting field data is strongly recommended, both from existing overhead lines and from especially designed devices.

Field ice data can be obtained by the following means:

1) Direct measurements of icing thickness or weight of samples taken from structures and line conductors. Ice samples fallen on the ground can be used, if consideration is given to the shape of initial ice accretion on conductors and to the fact that fallen pieces may represent only a fraction of the ice coating on the conductor.
2) Measurement on devices that simulate ice accretion on conductors. Devices currently used in some countries consist of simple tubes, rods or cable assemblies, 2–5 m above ground level in order to facilitate measurement.
3) Estimation of icing using conductor tension or the vertical component of weight at the attachment point at the support.
4) Estimation of icing using the conductor sag.

Ice loading data are important not only to establish design load criteria and their associated failure probabilities, but can also be useful in the planning stages of the transmission network and route selection of transmission lines.

Since very few countries have data on ice loadings at their disposal, and considering that it takes at least 10 years of field observation to acquire such a data base, it is strongly recommended that any utility planning a major line project, should undertake an ice observation programme without delay. Very often it will be possible to obtain the collaboration of the national weather services for the operation of instruments placed in standard meteorological stations.

However, any source of available and useful information should be collected and combined systematically in order to reduce, as much as possible, the time and cost of field measurements.

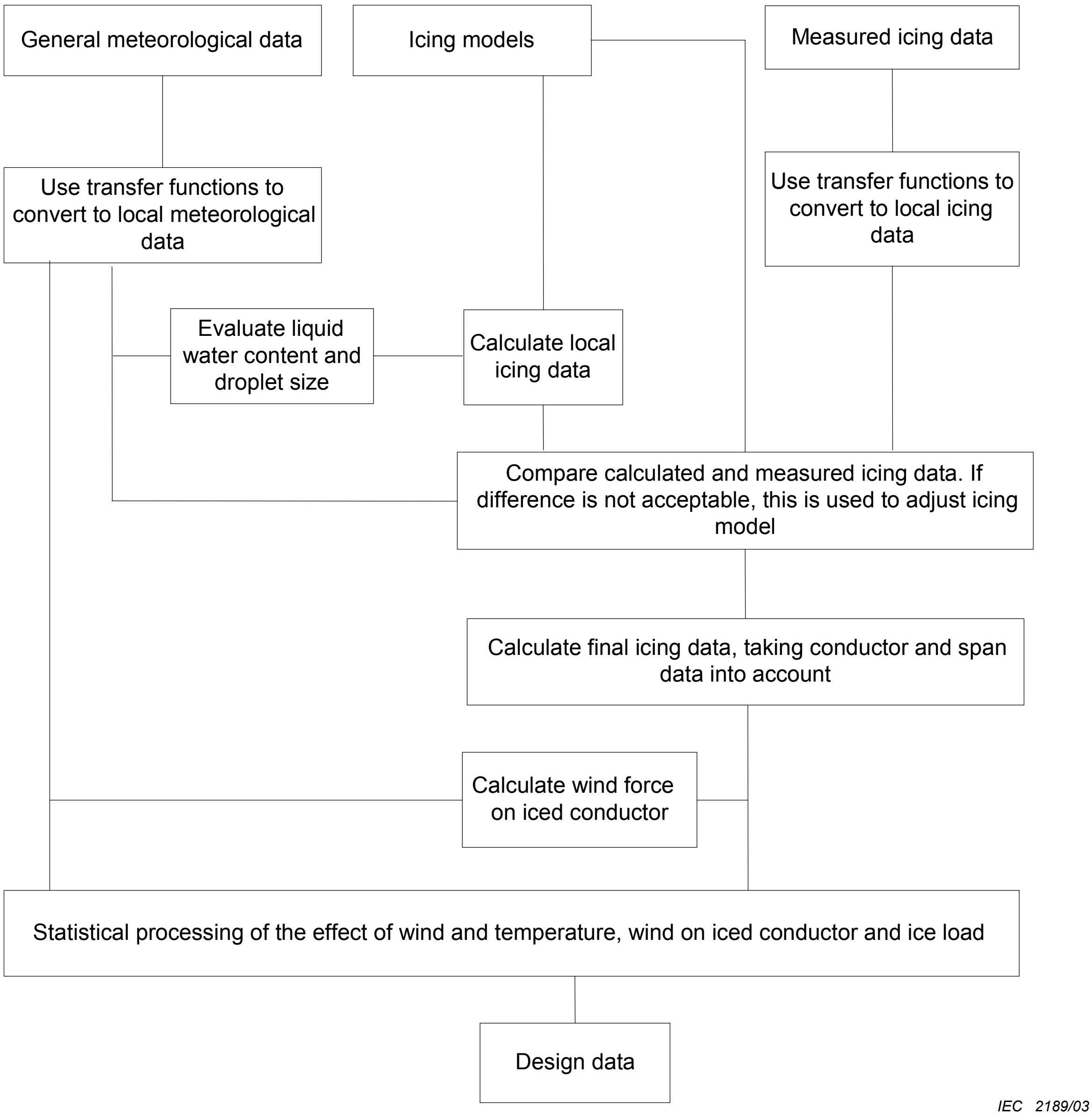


Figure A.10 – Strategy flow chart for utilizing meteorological data, icing models and field measurements of ice loads

IEC 61774 is a comprehensive standard covering all needs for meteorological data needed for overhead line design, and where measurements of icing is a significant part. Figure A.10 is taken from that standard. It demonstrates the strategy recommended in order to optimize the information that can be extracted from general meteorological data, icing models and separate measurements of ice loadings. Further guidelines on the selection of measuring sites, measuring devices as well as other instrumentation are also included in this standard.

Finally, it is recommended that the implementation of any major observation programme, as well as the analysis of the meteorological data, is conducted under the supervision of a professional meteorologist.

A.5.8 Ice data

A.5.8.1 Evaluation of information on ice loads

The available data on ice formation varies to a large extent. Depending on the available information and the number of years of observation, the following approaches for evaluation are recommended.

If records of yearly maximum ice loads during a period of at least 10 years are available, the mean value $\overline{g}$ is derived from the records of the yearly maximum ice load; the standard deviation σ_g is calculated or estimated according to Table A.12.

Table A.12 – Statistical parameters of ice loads

Number of years with observation n	Mean value $\overline{g}$	Standard deviation σ_g
$10 \le n \le 20$	$\overline{g}$	$0,5\ \overline{g} \le \sigma_g \le 0,7\ \overline{g}$
≥ 20	$\overline{g}$	$\sigma_g < 0,7\ \overline{g}$

In Table A.12, $\overline{g}$ is the calculated mean of the yearly maximum values g of ice load during the period concerned and σ_g is the calculated or estimated standard deviation.

If only the maximum value g_{max} of ice load during a certain number of years is available (no statistical data), the mean value $\overline{g}$ should be taken as 0,45 g_{max} and standard deviation σ_g as 0,5 $\overline{g}$.

A meteorological analysis model can be used to calculate values for yearly maximum ice loads during a certain number of years.

Sufficient data for using the statistical approach in this standard may be obtained by means of an analysis of available standard weather or climatological data over a period of 20 years or more, combined with at least five years of ice observation on the transmission line sites.

Information about the line sites which is necessary to validate and adjust the predicting model may be taken from past experience with existing transmission or distribution lines, from field observations in snowstorm sites or from the effect of icing on vegetation.

Such a predicting model can be rather simple or become sophisticated depending on terrain, local weather, number or types of collecting sites.

The results of this model analysis are used to find the mean value $\overline{g}$ and the standard deviation σ_g following the method given above.

A.5.8.2 Influence of height and conductor diameter

The factor K_d , conductor diameter, is given in 6.4.4.1 as well as the factor K_h , height z above ground. They can be approximated by the following formulae.

For in-cloud icing

$$K_d \sim 0,15\ d/30 + 0,85 \qquad \text{(A.52)}$$

NOTE The variation of in-cloud icing with height is very dependent on local topographical and climate conditions. Thus a specific climatological study is suggested to assess this variation for a line exposed to in-cloud icing.

and for precipitation icing

$$K_d \sim 0{,}35\ d/30 + 0{,}65 \quad \text{(A.53)}$$

$$K_h = 0{,}075 \times z\ /\ 10 + 0{,}925 \quad \text{(A.54)}$$

The above value of K_h for precipitation icing has been obtained using a simple icing model with a wind speed of about 25 km/h at 10 m and droplets fall speed of about 5 m/s.

A.5.8.3 The effect of icing on structures

Ice accretion on structures increases their vertical loads on the structure and may control the design of foundations and some support members.

The weight of ice can be calculated using the geometry of the support members and the relevant thickness of ice accretion. Alternatively, an approximation can be derived from the following table, where:

Ice thickness (mm)	15	25	30	35	40	45	50
Ratio of weight of ice to structure weight	0,57	1,00	1,23	1,48	1,73	2,00	2,28

A.6 Combined wind and ice loadings

A.6.1 Combined probabilities

The action of wind on ice-covered conductors involves at least three variables: wind associated with icing situations, ice weight and ice shape. These combined effects can result in both transversal and vertical loads. Direct measurements of these loads should, ideally, be the best approach but due to the difficulties and cost involved, such measurements are scarce and are not usually available.

Since it is possible to obtain independent observations of wind speed, ice weight and ice shape, it is proposed to combine these variables in such a way that the resulting load will have at least approximately the same return periods T as those adopted for each reliability level.

Combining the probabilities of correlated variables would, however, require the knowledge of the various interacting effects of these variables on the loadings. Assuming that maximum loads are most likely to be related to maximum values of individual variables (wind speed, ice weight and ice shape) a simplified method is proposed. A low probability value of a variable (index L) is combined with high probability (index H) values of the other two variables, as shown in Table A.13. In this method, a certain degree of independence between the different variables is accepted.

Table A.13 – Combined wind and ice loading conditions

Loading conditions	Ice weight	Wind speed	Effective drag coefficient	Density
Condition 1	g_L	V_{IH}	C_{IH}	δ_1
Condition 2	g_H	V_{IL}	C_{IH}	δ_1
Condition 3	g_H	V_{IH}	C_{IL}	δ_2

The high probability is considered to be the average of extreme yearly values, while the low probability of the variable is the one corresponding to a return period *T*.

A.6.2 Drag coefficients of ice-covered conductors

Field measurement is the best approach for the determination of the drag or lift coefficients of ice-covered conductors. However, at the current time of writing, very few such measurements exist. As a result, statistical distributions of drag or lift coefficients are not yet known.

As long as statistical data on the effective drag coefficients and densities are not available, it is suggested, in the absence of other experimental values, that the values given in Table A.14 should be used.

Table A.14 – Drag coefficients and density of ice-covered conductors

	Wet snow	Soft rime	Hard rime	Glaze ice
Effective drag coefficient C_{IH}	1,0	1,2	1,1	1,0
Density δ_1 (kg/m³)	600	600	900	900
Effective drag coefficient C_{IL}	1,4	1,7	1,5	1,4
Density δ_2 (kg/m³)	400	400	700	900

Annex B
(informative)

Application of statistical distribution functions to load and strength of overhead lines

B.1 General

The magnitude of climatic loads, their occurrence and the strength of components as well as use factors can be generally described by statistical distribution functions.

This annex describes the statistical characteristics of the phenomena to be represented and gives some proposals on the choice of distributions, which might fit more closely among those presented in Annex C.

To decide on an adequate distribution it is preferable to carry out tests on the fitting of the available data what ever the phenomenon.

These tests should begin with screening of data and exclusion of outlying values for various reasons: recording mistakes, inadequate measurements, interpretation difficulties, etc.

This step establishes a reliable database on which relevant curve fittings could be performed to find an adequate statistical representation of the phenomenon. Various software can perform such fittings and in addition, the significance level of adequacy can be calculated according to classical tests for acceptance of fit: χ^2 (Chi squared), Kolmogorov-Smirnov, etc.

The conclusions might differ from that proposed hereafter, but they should be preferred if they offer a better fit to the actual data series.

B.2 Climatic loads

B.2.1 Wind velocities and wind loads

The distribution of yearly maximum wind velocities is usually described by a Gumbel distribution; an example being given in Figure B.1:

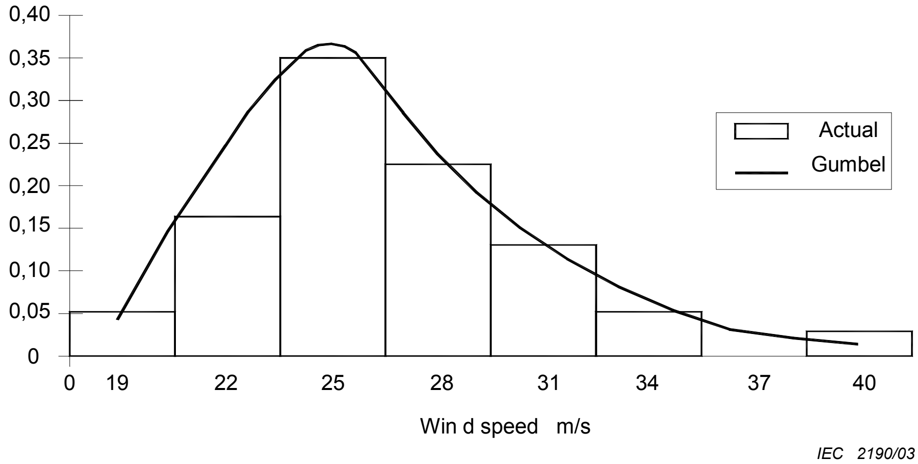


Figure B.1 – Fitting of Gumbel distribution with wind data histogram

The mathematical simplicity of the Gumbel distribution (defined by only two variables) allows an easy curve fitting by referring to Clause C.3. However, some approaches and building codes prefer to analyse the wind pressure (instead of speed) according to a Gumbel law. This is the basic assumption of EUROCODE.

In addition, it should be noted that many countries offer wind maps covering their territories, giving usually the 10 min wind speed with 50 years return period. In the USA, the 3 s wind speed has recently been adopted as the statistical variable of wind speed.

Where available, these maps should be used directly.

An analysis of meteorological data has shown that the distribution of annual maximum wind velocities or ice loads and ice thicknesses can be represented, with good approximation, by an extreme value distribution law (Gumbel Type I).

The basic formula for the Gumbel Type I cumulative distribution function has the form:

$$F(x) = \exp\{-\exp[-a \times (x - u)]\} \tag{B.1}$$

where

$$a = C_1 / \sigma \quad \text{and} \quad u = \bar{x} - C_2 / a \tag{B.2}$$

This formula expresses the probability $F(x)$ that a random value will be less than a value x in a distribution with a mean value $\bar{x}$ and a standard deviation σ.

The parameters C_1 and C_2 depend on the number of years (n) with observations and are given in Table B.1. For calculation of C_1 and C_2 see Clause C.4 and Table C.1.

The general form of Equation (B.1) thus becomes:

$$F(x) = \exp\left\{-\exp\left[-\frac{C_1}{\sigma}\left(x - \bar{x} + \frac{C_2 \times \sigma}{C_1}\right)\right]\right\} \tag{B.3}$$

and in the case where $n \approx \infty$

$$F(x) = \exp\left\{-\exp\left[-\pi\left(x - \bar{x} + 0{,}45\sigma\right)/\left(\sigma \times \sqrt{6}\right)\right]\right\} \tag{B.4}$$

Hence, the probability $P(x)$ that the observed value will be higher than x during one year is:

$$P(x) = 1 - \exp\left\{-\exp\left[-\pi\left(x - \bar{x} + 0.45\sigma\right)/\left(\sigma \cdot \sqrt{6}\right)\right]\right\} \tag{B.5}$$

As a simplification the return period T of the value x is given by:

$$T = \frac{1}{P(x)} \tag{B.6}$$

By rearranging the Formulae (B.3) and (B.6) the following is obtained:

$$x = \bar{x} - \frac{C_2\sigma}{C_1} - \frac{\sigma}{C_1}\left[\ln\left(-\ln\left(1 - 1/T\right)\right)\right] \tag{B.7}$$

Formula (B.7) gives the value x with a return period T as a function of $\bar{x}$, C_1 and C_2.

If $n \approx \infty$, then:

$$x_\infty = \bar{x} - 0.45\sigma - \frac{\sigma\sqrt{6}}{\pi}\left[\ln\left(-\ln\left(1 - 1/T\right)\right)\right] \tag{B.8}$$

From Equations (B.7) and (B.8) the ratio x ($n < \infty$) / x ($n = \infty$) follows, which is defined as $K_{n.}$

$$K_n = x_n/x_\infty = \left\{\bar{x} - \frac{C_2(n)\cdot\sigma}{C_1(n)} - \frac{\sigma}{C_1(n)}\left[\ln(-\ln(-1/T))\right]\right\} /$$

$$\left\{\bar{x} - 0{,}45\sigma - \frac{\sigma\sqrt{6}}{\pi}\left[\ln(-\ln(1-1/T))\right]\right\} \tag{B.9}$$

From Equation (B.8) the parameter $K_{\sigma x}$ is obtained, which applies to an infinite number of observations

$$K_{\sigma x} = x_\infty/\bar{x} = 1\text{-}0{,}45\,\frac{\sigma}{\bar{x}} - \frac{\sigma}{\bar{x}}\frac{\sqrt{6}}{\pi}\left[\ln(-\ln(1-1/T))\right]$$

or (B.10)

$$K_{\sigma x} = 1 - v_x\left\{0{,}45 - \frac{\sqrt{6}}{\pi}\left[\ln(-\ln(1-1/T))\right]\right\}$$

Finally, Equations (B.7), (B.9) and (B.10) yield

$$x/\overline{x} = K_{\mathrm{n}} \times K_{\sigma x} = 1 - \frac{v_x}{C_1}\left[C_2 + \ln\left(-\ln\left(1 - 1/T\right)\right)\right] \tag{B.11}$$

The values $x/\overline{x}$ as obtained from Equation (B.11) are given in Table B.1 for various return periods T and number of years of observation n.

Wind velocities cannot be directly combined with strength of supports because the loadings depend on the square of wind velocities.

$$L_{\mathrm{w}} = kV^2 \tag{B.12}$$

The factor k depends on various parameters such as the conductor diameter, drag coefficient, gust factors, etc. For line design it may be necessary to know the statistical characteristics of wind loads. An appropriate distribution can be obtained by assuming

$$\overline{L} \sim \mathrm{k}\overline{\mathrm{V}}^{\,2} \text{ and } \sigma_{\mathrm{L}}/\overline{L} \sim 2\sigma_{\mathrm{V}}/\overline{V} \tag{B.13}$$

These relations can be proved by carrying out examples.

With $x = V$; $\overline{x} = \overline{V}$ and $\sigma = \sigma_{\mathrm{V}}$ from (B.3) results

$$F(v) = \exp\left\{-\exp\left[-\frac{C_1}{\sigma_v}\left(V - \overline{V} + \frac{C_2}{C_1}\sigma_v\right)\right]\right\} \tag{B.14}$$

From Equation (B.13)

$$\overline{V} = \sqrt{\overline{L}/k} \text{ and } \sigma_{\mathrm{V}} = 2\sigma_{\mathrm{L}} \cdot \overline{V}/\overline{L}$$

follow. Therefore, Equation (B.14) yields

$$F(L) = \exp\left\{-\exp\left[-\frac{C_1}{\sigma_L}\left(2\sqrt{L\overline{L}} - 2\overline{L} + \frac{C_2}{C_1}\sigma_L\right)\right]\right\} \tag{B.14a}$$

Matching loads and strength can be obtained by using twice the standard deviation of wind velocities.

Table B.1 – Ratios of $x / \bar{x}$ for a Gumbel distribution function, *T* return period in years of loading event, *n* number of years with observations, v_x coefficient of variation

COV / *T*	Reliability level 1 50 years						Reliability level 2 150 years						Reliability level 3 500 years					
v_x / *n*	10	15	20	25	50	∞	10	15	20	25	50	∞	10	15	20	25	50	∞
0,05	1,18	1,17	1,16	1,15	1,14	1,13	1,24	1,22	1,21	1,21	1,19	1,17	1,30	1,28	1,27	1,26	1,24	1,22
0,075	1,27	1,25	1,24	1,23	1,22	1,19	1,36	1,33	1,32	1,31	1,29	1,26	1,45	1,42	1,40	1,39	1,37	1,33
0,10	1,36	1,33	1,32	1,31	1,29	1,26	1,48	1,44	1,42	1,41	1,38	1,36	1,60	1,56	1,54	1,52	1,49	1,44
0,12	1,43	1,40	1,38	1,37	1,35	1,31	1,57	1,53	1,51	1,49	1,46	1,41	1,72	1,67	1,64	1,62	1,59	1,53
0,15	1,54	1,50	1,48	1,46	1,43	1,39	1,71	1,66	1,63	1,62	1,58	1,52	1,90	1,84	1,80	1,78	1,73	1,66
0,16	1,57	1,53	1,51	1,49	1,46	1,41	1,76	1,70	1,68	1,66	1,61	1,55	1,96	1,89	1,86	1,83	1,78	1,70
0,20	1,72	1,66	1,64	1,62	1,58	1,52	1,95	1,88	1,84	1,82	1,77	1,69	2,20	2,12	2,07	2,04	1,98	1,88
0,25	1,90	1,83	1,79	1,77	1,72	1,65	2,19	2,10	2,05	2,03	1,96	1,86	2,51	2,40	2,34	2,30	2,22	2,10
0,30	2,08	2,00	1,95	1,93	1,87	1,78	2,43	2,32	2,27	2,23	2,15	2,04	2,81	2,68	2,61	2,56	2,46	2,32
0,35	2,26	2,16	2,11	2,06	2,01	1,91	2,66	2,54	2,48	2,44	2,34	2,21	3,11	2,96	2,87	2,82	2,71	2,54
0,40	2,43	2,33	2,27	2,24	2,16	2,04	2,90	2,76	2,69	2,64	2,54	2,36	3,41	3,23	3,14	3,08	2,95	2,76
0,45	2,61	2,49	2,43	2,39	2,30	2,17	3,14	2,98	2,90	2,85	2,73	2,55	3,71	3,51	3,41	3,34	3,20	2,98
0,50	2,79	2,66	2,59	2,54	2,44	2,30	3,38	3,20	3,11	3,05	2,92	2,73	4,01	3,79	3,68	3,60	3,44	3,20
0,55	2,97	2,83	2,75	2,70	2,59	2,43	3,61	3,42	3,32	3,26	3,11	2,90	4,31	4,07	3,94	3,86	3,68	3,42
0,60	3,15	2,99	2,91	2,85	2,73	2,56	3,85	3,64	3,53	3,46	3,30	3,07	4,61	4,35	4,21	4,12	3,93	3,64
0,65	3,33	3,16	3,07	3,01	2,88	2,68	4,09	3,86	3,74	3,67	3,50	3,25	4,91	4,63	4,48	4,38	4,17	3,86

B.2.2 Temperatures

The yearly minimum temperatures can also be fitted by a Gumbel distribution; an example is given in Figure B.2 below:

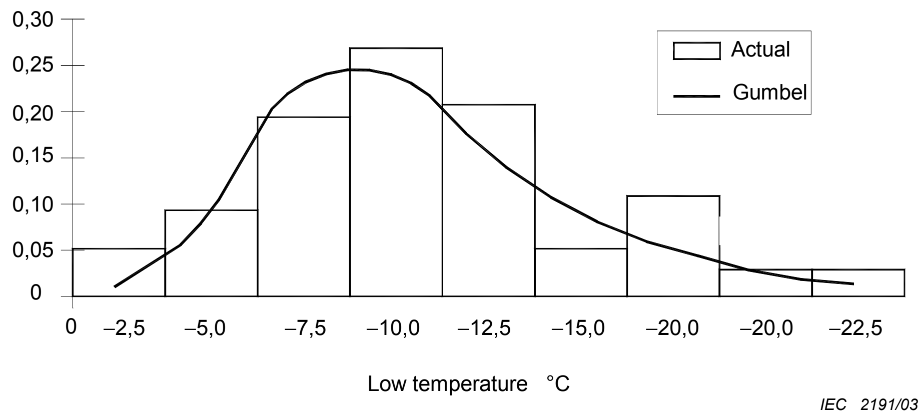


Figure B.2 – Fitting of Gumbel distribution with yearly minimum temperature histogram

Again, when temperature maps are available, they should be used.

B.2.3 Ice loads

The yearly maximum ice loads show different patterns.

In certain regions there are many years without any icing event and a few years with severe overloads. Such behaviour cannot be described easily by a Gumbel law; an example is given in Figure B.3:

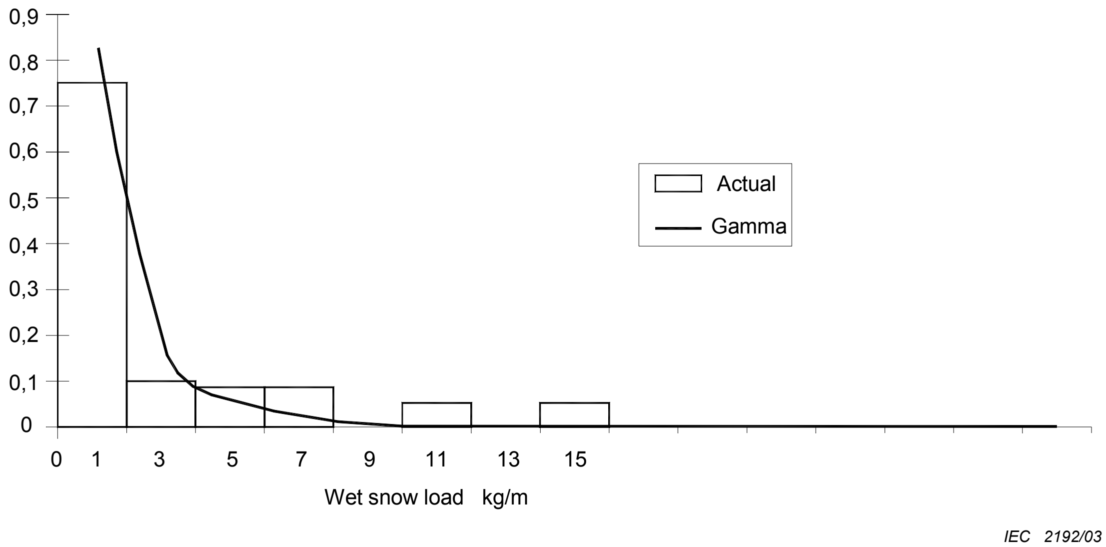


Figure B.3 – Fitting of Gamma distribution with ice load histogram

While the Gumbel distribution is widely used for ice loads, some studies suggest that the Gamma distribution is often suited to represent such a phenomenon, particularly when ice is not a yearly phenomenon.

To ease the curve fitting, this study revealed that the parameter p_1 can be taken slightly lower than 0 in order to get a finite value for the density value *f(0)*, for instance $p_1 \approx -0{,}03$.

A physical explanation of this special pattern could lie in the threshold effect of temperature on these icing events.

For in-cloud icing, it is necessary that the temperature should be below 0 °C. This means that for regions with a relatively mild climate, all the conditions for ice accretion might often be attained except for one – adequate temperature. This might explain the rarity of the event.

For wet snow, there are two temperature thresholds, usually –1 °C and +2 °C, which, according to observations, makes the phenomenon even more rare than in-cloud icing.

Nevertheless, these events are severe enough when they occur so they cannot be neglected.

In regions with a more rugged climate, some in-cloud icing patterns can come closer to that of a Gumbel distribution and this distribution might be recommended in specific conditions such as in the example presented in Figure B.4.

In this case, it is supposed that the adequate temperature conditions are met more often. The effect of increased ice loads due to altitude should also be mentioned here.

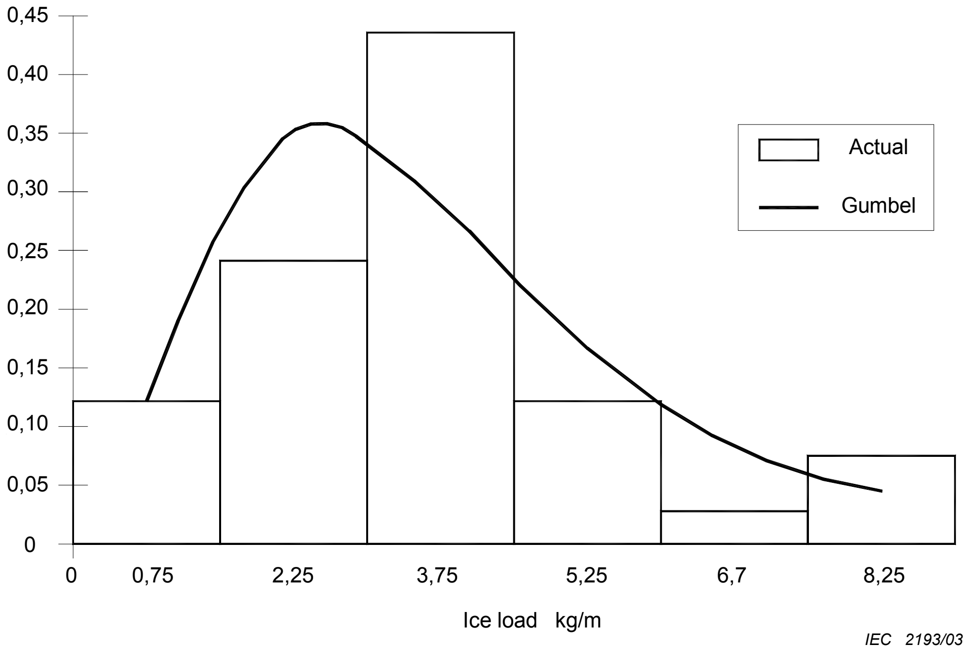


Figure B.4 – Fitting data from in-cloud icing with Gumbel distribution

B.3 Strength of components

B.3.1 Lattice supports

Studies dealing with the results of support tests obtained from testing stations, found that it was possible to statistically represent their strength, as illustrated in Figure B.5.

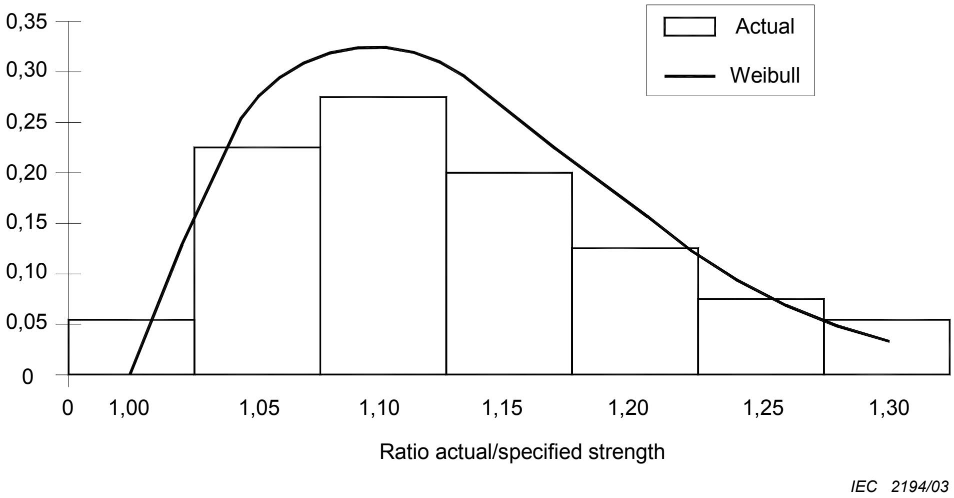


Figure B.5 – Fitting of Weibull distribution with strength data of lattice supports

Different sets of data concerning lattice supports showed the same representation by a threshold distribution (Weibull or Gamma). Moreover, the threshold parameter (p_1) is always equal to 1: all the tested supports met the specified strength. The difference in data sets lies in varying dispersion, measured by coefficient of variation COV, probably due to different design criteria.

However, for design of a single component the use of a normal or log-normal distribution will not result in essential discrepancies. For design of components in series or simultaneously subjected to maximum loads, the use of a distribution with a lower threshold (log-normal of Weibull distribution) is more advisable.

As guidance, if no specific data are available, a Weibull distribution with the parameters given in Table B.2 can be used.

Table B.2 – Parameters of Weibull distribution

COV ***v***	**0,075**	**0,10**
m	1,13	1,13
p_1 (threshold)	1	1
p_2 (scale)	0,145	0,137
p_3 (shape)	1,57	1,155
(10 %)R / R_{sp} [a]	1,03	1,02
(5 %)R / R_{sp}	1,02	1,01
(2 %)R / R_{sp}	1,01	1,00
[a] R_{sp} is the specified strength which is equal to the threshold p_1.		

An interesting consequence of the choice of a threshold distribution is that the characteristic strengths R_c (for a 2 %, 5 % or 10 % exclusion limit) are very close to the characteristic value. Therefore, for simplification, just the well known rated value could be retained.

B.3.2 Other manufactured components

The basic conclusions drawn from experimental study of lattice supports suggest the adequacy of a threshold distribution with a threshold parameter equal to 1.

They can be extended to any carefully manufactured component with a good quality assurance. Physically, it means that every effort is made during the manufacturing process to reach at least the rated strength. This may be applied to conductors, fittings, insulators, steel poles, etc.

B.3.3 Components with no well defined strength

Contrarily to the components described in B.3.2, the strength of some components is based on the characteristics of natural materials. This applies specifically to wooden poles as well as foundations whose strengths are determined by the characteristics of the surrounding soil.

Due to the variation of their behaviour under loading it is not possible to give proven recommendations for their statistical description. The only suggestion could be to use a normal distribution with no definite lower limit.

B.4 Effect of span variation on load-strength relationship – Calculation of span use factor

B.4.1 General

Assuming F is the force resulting from climatic actions applied to the maximum span L_{max}, then the force on a support with a span L_i is in linear systems equal to $F \times L_i/L_{max}$. In the case of wind loads and a large difference between L_i and L_{max} there is small non-linearity introduced by the variation of gust factor with spans. This has little influence on the reliability because the latter is controlled by spans near maximum values where the span/load relation can be assumed to be linear. The ratio of L_i/L_{max} is a random variable called use factor U. The use factor has an upper bound of 1,0 and a lower bound typically equal to 0,4. From the analysis of lines designed according to limit load concept, it has been found that the use factor can be assumed to have a Beta distribution function. A general description of the Beta function is found in Clause C.7.

The use factor depends mainly on three variables: the number of types of suspension supports available for spotting, the category of the terrain and the constraints on support locations. For example, if every support in a line is custom-designed for the exact span at each location, the use factor will be equal to 1,0. While if only one suspension support type is used in a line located in mountainous terrain, the average use factor will be significantly less than 1, typically 0,60 to 0,75.

The use factor variation was found to have predictable patterns and statistical parameters $\overline{U}$ and σ_U could be known with sufficient accuracy if the number of suspension support types, terrain and spotting constraints were known.

In Tables B.3 and B.4, typical mean values $\overline{U}$ and standard deviation σ_u are given. Note that $\overline{U}$ can be derived from the design criteria of tangent supports if the average span of the transmission line is known, because of the following relation:

Average span = Average wind span = Average weight span

Thus, the average wind use factor $\overline{U}_{\text{wind}}$, can be calculated from:

$$\overline{U}_{\text{wind}} = \frac{\text{Average wind span}}{\text{Design wind span}} = \frac{\text{Average span}}{\text{Design wind span}} \tag{B.15}$$

Similarly,

$$\overline{U}_{\text{weight}} = \frac{\text{Average weight span}}{\text{Design weight span}} = \frac{\text{Average span}}{\text{Design weight span}} \tag{B.16}$$

In Tables B.3 and B.4 the lower bound of the use factor is assumed to be 0,4 and the upper bound is 1,0. Furthermore, the following codes are used:

– codes for terrain type:
  - a flat,
  - b rolling, hilly,
  - c mountainous;
– codes for constraints on support locations:
  - 1 no special constraints,
  - 2 constraints on support locations.

Thus a1 means flat terrain without special constraints.

NOTE Constraints imply that the freedom to position the supports on the most economical locations is reduced because of special considerations such as environment or location of roads, private properties, lines, etc.

In the absence of specific data, the statistical parameters of U identified in Tables B.3 and B.4 can be considered typical for describing span variation of common transmission lines.

Table B.3 – Statistical parameters $\overline{U}$ and σ_u of wind span variation

Terrain and constraints	**a1**		**b1, a2**		**b2, c1**		**c2**	
	$\overline{U}$	σ_u	$\overline{U}$	σ_u	$\overline{U}$	σ_u	$\overline{U}$	σ_u
Number of suspension support types								
1	0,95	0,05	0,85	0,10	0,75	0,15	0,55	0,20
2	≈ 1,0	0,00	0,95	0,05	0,85	0,10	0,65	0,15
3	≈ 1,0	0,00	≈ 1,0	0,00	0,95	0,05	0,75	0,10

Table B.4 – Statistical parameters $\overline{U}$ and σ_u of weight span variation

Terrain and constraints	a1		b1, a2		b2, c1		c2	
	$\overline{U}$	σ_u	$\overline{U}$	σ_u	$\overline{U}$	σ_u	$\overline{U}$	σ_u
Number of suspension support types								
1	0,85	0,05	0,75	0,10	0,65	0,15	0,50	0,20
2	0,95	0,03	0,85	0,05	0,75	0,10	0,60	0,15
3	≈ 1,0	0,00	≈ 1,0	0,00	0,85	0,05	0,70	0,10

B.4.2 Effect of use factor on load reduction and its calculation

As discussed in 5.2.3, the fact that all supports are not used with their maximum spans contributes to an increase in reliability.

When the designer aims to design for a target reliability he can, provided that sufficient data on span variation is available, reduce the design loads on supports by a factor γ_u <1 and achieve more economical lines.

The reduction factors coefficient γ_u, generally called use factor coefficient, can be calculated from the statistical methods detailed hereafter (see 5.2.3).

B.4.2.1 Calculation of use factor coefficient by statistics and simulation

For calculation of the use factor coefficient simulation by the Monte-Carlo method based on statistical distributions can be adopted.

- Start with a given function of wind speed V (assume Gumbel function), where: v_w = 0,12; 0,16; 0,20 (coefficient of variation of wind speed.)
- Consider different cases of use factor curves (assume Beta function), where: $\overline{U}$ = 0,95, σ_u = 0,05 and $\overline{U}$ = 0,85, σ_u = 0,10, etc.
- Calculate statistical parameters of applied load $Q' = W^2U$. This can be done by Monte-Carlo simulation or by numerical integration. For small values of σ_u, approximate methods can be used.
- Calculate Q_{50} from $Q = W^2$ (using Gumbel tables). This is the load without the effect of use factor coefficient.
- The use factor coefficient γ_u is calculated from the ratio Q'_{50}/Q_{50}.

This procedure is demonstrated by a numerical example:

Input data: $\overline{W}$ = 1,0, v_w = 0,12 (Gumbel)

$\overline{U}$ = 0,85, σ_u = 0,10 (Beta)

Calculations: $W_{50} = 1{,}31\ \overline{W}$ (from Gumbel tables), see B.2.1, Table B.1

$Q_{50} = \text{K} \times W_{50}^2 = 1{,}72$ K (where K is a constant).

Calculate the probability density function of $Q' = W^2U$. From the new Q', we obtain Q'_{50} = K × 1,48 (value obtained by numerical integration).

Thus $$\gamma_u = \frac{Q'_{50}}{Q_{50}} = \frac{1{,}48K}{1{,}72K} = 0{,}86$$

This exercise is repeated for T = 150 years, 500 years as well as for v_w = 0,16 and 0,20. All resulting values of γ_u are found to be in the range of 0,86 to 0,89.

B.4.2.2 Approximation by the method of central moments

The method of central moments can be adopted to determine use factor coefficients approximately. Based on the data used in the numerical example above, the method is as follows:

$$Q' = \overline{W}^{\,2} \times \overline{U} = 1{,}0^2 \times 0{,}85 = 0{,}85$$

$$v_Q{}^2 = (2v_w)^2 + v_U{}^2 = 0{,}0576 + 0{,}0138 \Rightarrow v_{Q'} = 0{,}267$$

Q'_{50} (T = 50 years) = Q′ with a 2 % probability. It is equal to 2,05 standard deviations from mean value for a normal distribution.

$$Q'_{50} = 0{,}85\ (1 + 2{,}05 \times 0{,}267) = 1{,}32$$

Q_{50} (T = 50 years), without the effect of U:

$Q \quad = \quad W^2$

$Q \quad = \quad 1{,}0;\ v_Q = 2 \times 0{,}12 = 0{,}24$

$Q_{50} \quad = \quad 1{,}0\ (1 + 2{,}05 \times 0{,}24) = 1{,}49$

$$\gamma_u = \frac{1{,}32K}{1{,}49K} = 0{,}88$$

It is noted that approximate calculations do not yield good estimates of load Q_T but give acceptable results for γ_u.

B.4.3 Effect of number of supports on use factor coefficients

When the maximum intensity of a climatic load event covers a large number of supports *(N),* the impact of the use factor variation is then changed.

If the strength variation is very low or can be neglected, then the most critically loaded support amongst the *N* supports will be the one having the largest span or highest use factor coefficient.

In order to find γ_u for *N* supports, we have to consider in the previous calculation a new curve $min_N U$.

When *N* is very large and the strength variation is assumed negligible, γ_u is very close to 1,0.

The physical explanation of γ_u increasing up to 1,0 is simple: if the maximum intensity of a loading event covers a large space, then it will likely act on the support having the longest span.

As indicated, the above conclusion can be made only if the strength variation of supports is neglected compared to that of spans. However, in well-optimized lines, the use factor coefficient of variation can be low and it is important to establish the joint effect of use factor and strength variation.

Two methods may be used for this purpose and lead to a similar result: the first one using the general relation *Q' U < R* and the second *Q < R/U*. The second method will be detailed hereafter.

The strength used in the design equation is (10 %) *R*. With the influence of the use factor coefficient, this value becomes (10 %) *R/U*. With *N* supports subjected to maximum load intensity, the design strength becomes (10 %) $\min_N$ *(R/U).*

Thus the influence of the use factor can be quantified by:

$$\gamma_U = \frac{(10\,\%)\min_N R}{(10\,\%)\min_N R/U} \qquad \text{(B.17)}$$

The numerator in Equation (B.17) has been discussed in B.4.3. The denominator can be obtained by the same methods, either by simulation or by numerical integration. Table B.5 outlines some results obtained by Monte-Carlo simulation.

Table B.5 – Values of use factor coefficient γ_u as a function of *U* and *N* for v_R = 0,10

	N	**1**	**2**	**5**	**10**	**20**	**50**	**100**
U	σ_u							
0,95	0,05	0,96	0,96	0,97	0,97	0,97	0,97	0,97
0,85	0,10	0,90	0,90	0,91	0,92	0,94	0,94	0,94
0,80	0,15	0,89	0,90	0,91	0,91	0,92	0,93	0,94

From Table B.5 we notice that the use factor coefficient γ_u increases very slightly with increasing number of supports *N* and can be taken as nearly constant. This result can be explained by the fact that the support having the largest span is not necessarily the one with the lowest strength. Strength variation and spans are two independent random variables.

In order to better illustrate the calculations of this table, an approximate method to derive γ_u is used:

Input data: $\overline{R}$ = 1,0; v_R = 0,10; $\overline{U}$ = 0,95; σ_u = 0,05 (v_u = 0,053); *N* = 10

First, (10 %) $\min_N R$ is calculated:

From Equation (A.23) and Table B.5:

$$(10\ \%)\ \min_N R = \Phi_N \times 10\ \%\ R = 0{,}89 \times (1 - 1{,}28 \times 0{,}10) = 0{,}77$$

can be obtained. In order to calculate (10 %) $\min_N R/U$, the statistical parameters of $R' = \overline{R}/\overline{U}$ have to be determined:

$$R' = \frac{\overline{R}}{\overline{U}} = \frac{1{,}00}{0{,}95} = 1{,}053$$

$$v_R' = \sqrt{\left(v_R^2 + v_U^2\right)} = \sqrt{\left(0{,}10^2 + 0{,}053^2\right)} = 0{,}113$$

With N = 10, the 10 % exclusion limit corresponds to 2,31 standard deviations (see 2.3.5), therefore:

$$(10\ \%)\ \min_N R' = \overline{R}'\ (1 - 2{,}31\ v_R') = 1{,}053\ (1 - 2{,}31 \times 0{,}113) = 0{,}78$$

$$\gamma_u = \frac{(10\%)\min_N R}{(10\%)\min_N R'} = \frac{0{,}77}{0{,}78} = 0{,}99 \text{ (compared to 0,97 from Table B.5).}$$

It is noted that because of the discrete nature of U, approximate methods do not always give accurate results. The approximation discussed above is acceptable when v_u is much smaller than v_R and N <10.

B.4.4 Generalized results for use factor coefficients γ_u

Values of use factor coefficient γ_u similar to those given in Table B.5 were obtained for other v_R values (0,05, 0,075, 0,10, 0,15, 0,20 and 0,25). These values were found almost constant, especially when the variation of the use factor coefficient is low. Table B.6 summarises results of γ_u.

Table B.6 – Use factor coefficient γ_u for different strength coefficients of variation

$\overline{U}$	σ_u	N	Strength COV 0,05	Strength COV 0,10	Strength COV 0,15	Strength COV 0,20	Recommended value of γ_u
0,95	0,05	1	0,97	0,96	0,96	0,96	
		10	0,97	0,97	0,96	0,97	0,95
		100	0,98	0,97	0,97	0,97	
0,85	0,10	1	0,92	0,90	0,88	0,87	
		10	0,94	0,92	0,89	0,88	0,90
		100	0,96	0,94	0,90	0,92	
0,80	0,15	1	0,92	0,89	0,86	0,84	
		10	0,94	0,91	0,88	0,87	0,88
		100	0,96	0,94	0,90	0,88	
0,75	0,10	1	0,83	0,81	0,79	0,78	
		10	0,87	0,83	0,80	0,78	0,83
		100	0,88	0,84	0,81	0,78	

As discussed earlier, span dispersion can either increase strength by a factor of $1/\gamma_u$, or decrease applied loads, which are affected by the span lengths, by γ_u. The values of γ_u, if taken equal to 1,0 during the design of supports, will lead to a reliability higher than expected.

In cases where the considered supports will be used in different transmission lines where use factor coefficients can vary, it could be acceptable to use as a conservative value, the highest γ_u, or even γ_u = 1,0 which will lead to some over design. In the latter case, it is advisable to recheck the coordination of strength that may now be altered.

Annex C
(informative)

Statistical distribution and their application in probabilistic design of transmission lines

C.1 Classical statistical distributions

This annex presents the distribution functions that can be of interest to implement the reliability concepts presented in this standard. These distribution functions may be used to represent either a load having a meteorological origin or a component's strength.

In all the functions described hereafter, the parameters are defined depending on the characteristics:

m mean value of observations,

σ standard deviation of observations.

C.2 Normal distribution (Gaussian distribution)

C.2.1 General format

Probability density function:

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x-m}{\sigma}\right)^2\right], \sigma > 0 \tag{C.1}$$

Cumulative distribution function:

$$F_{(x)} = \frac{1}{\sigma\sqrt{2\pi}} \int_{-\infty}^{x} \exp\left[-\frac{1}{2}\left(\frac{x-m}{\sigma}\right)^2\right] dx \tag{C.2}$$

C.2.2 Standardized format

Variable change for the standardized form:

$$u = \frac{x-m}{\sigma} \tag{C.3}$$

Probability density function:

$$f_{(u)} = \frac{1}{\sqrt{2\pi}} \exp\left[-\frac{u^2}{2}\right] \tag{C.4}$$

Cumulative distribution function:

$$F_{(u)} = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{u} \exp\left[-\frac{u^2}{2}\right] du \tag{C.5}$$

There is no closed analytical presentation for the cumulative distribution, but an approximation is given below.

Approximate cumulative distribution:

$$F_{(u)} = 1 - f_{(u)}(b_1 t + b_2 t^2 + b_3 t^3 + b_4 t^4 + b_5 t^5) \quad \text{(C.6)}$$

where

b_1 = 0,319381530,

b_2 = -0,356563782,

b_3 = 1,781477937,

b_4 = -1,821255978,

b_5 = 1,330274429 and

t = 1 / (1 + 0,2316419u) (C.7)

$f_{(u)}$ = probability density function

Relation to return periods T and exclusion limit e

$$F_T\,(u_T) = 1 - 1\,/\,T \quad \text{(C.8)}$$

$$F_e\,(u_e) = e\,/\,100 \quad \text{(C.9)}$$

where T is in years, and e is in per cent (%).

While there is no analytical form for cumulative distribution, it is more convenient to refer to classical tables of which an extract is given hereafter:

T (years)	F_T	u_T
50	0,9800	2,0537
150	0,9933	2,4758
500	0,9980	2,8782

e (%)	F_e	u_e
2	0,02	–2,0537
5	0,05	–1,6449
10	0,10	–1,2816

Relation of variable $X(T)$ or $X(e)$ having a given return period T or an exclusion limit e to mean value m and standard deviation σ or coefficient of variation $v = \sigma/m$:

$$X(T) = m + u(T)\sigma \;=\; m\,(1 + u(T)v)$$

$$X(e) = m + u(e)\sigma \;=\; m\,(1 + u(e)v)$$

Another approximation can be used for Equation (C.5):

then $F_{(x)} \cong 0{,}6931\,\exp\left[-\left(\frac{9x-8}{14}\right)^2\right]$ valid for $x \leq 0$, in the range (–4, 0)

and $x \cong \frac{1}{9}\left[8 - 14\sqrt{\ln\left(\frac{0{,}6931}{F_{(x)}}\right)}\right]$ valid for $F_{(x)} \leq 0{,}5$

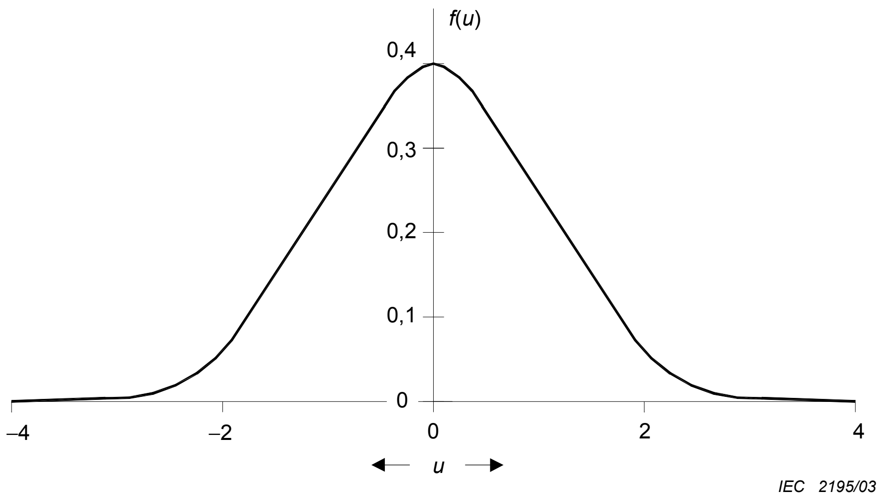


Figure C.1 – Probability density function of standardized normal distribution

C.3 Log-normal distribution

The log-normal distribution is defined as a distribution where $\ln(x\text{-}p_1)$ follows a normal distribution.

C.3.1 General format

Probability density function:

$$f_{(x)} = \frac{1}{p_2(x-p_1)\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\ln(x-p_1)-p_3}{p_2}\right)^2\right], \qquad \text{(C.10)}$$

$$x > p_1, p_2 > 0, p_3 > 0$$

Cumulative distribution function:

$$F_{(x)} = \frac{1}{p_2\sqrt{2\pi}} \int_{p_1}^{x} \frac{1}{x-p_1} \exp\left[-\frac{1}{2}\left(\frac{\ln(x-p_1)-p_3}{p_2}\right)^2\right] dx \qquad \text{(C.11)}$$

C.3.2 Standardized format

Variable change for application of standard format (Equations C.1, C.4 and C.5):

$$u = \frac{\ln(x - p_1) - p_3}{p_2}, \tag{C.12}$$

Probability density function:

$$f_{(u)} = \frac{1}{\sqrt{2\pi}} \exp\left[-\frac{u^2}{2}\right] \tag{C.13}$$

Cumulative distribution function:

$$F_{(u)} = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{u} \exp\left[-\frac{u^2}{2}\right] du \tag{C.14}$$

The standard format is in fact identical to the one of the normal distribution (Equation C.5) as a consequence of the log-normal distribution definition.

Relation to return period *T* and exclusion limit *e*

The numerical values of *u(T)* or *u(e)* are the same as for the normal distribution, see under Equation C.1.

Relation of variable *X(T)* and *X(e)* having a given return period *T* or an exclusion limit *e* to mean value *m* and standard deviation *σ*.

$$X(T) = p_1 + \exp\left(p_3 + u\,(T)\,p_2\right) \tag{C.15}$$

$$X(e) = p_1 + \exp\left(p_3 + u\,(e)\,p_2\right) \tag{C.16}$$

Values of parameters:

$$p_2 = \sqrt{\ln\left[1 + \frac{\sigma^2}{(m - p_1)^2}\right]} \tag{C.17}$$

$$p_3 = \ln\left[\frac{(m - p_1)^2}{\sqrt{(m - p_1)^2 + \sigma^2}}\right] \tag{C.18}$$

Another consequence of the specific definition of this distribution is the significance of the parameters:

p_3 is the mean value of the variable $\ln(x\text{-}p_1)$,

p_2 is the standard deviation of the variable $\ln(x\text{-}p_1)$,

p_1 is the lower threshold of the distribution.

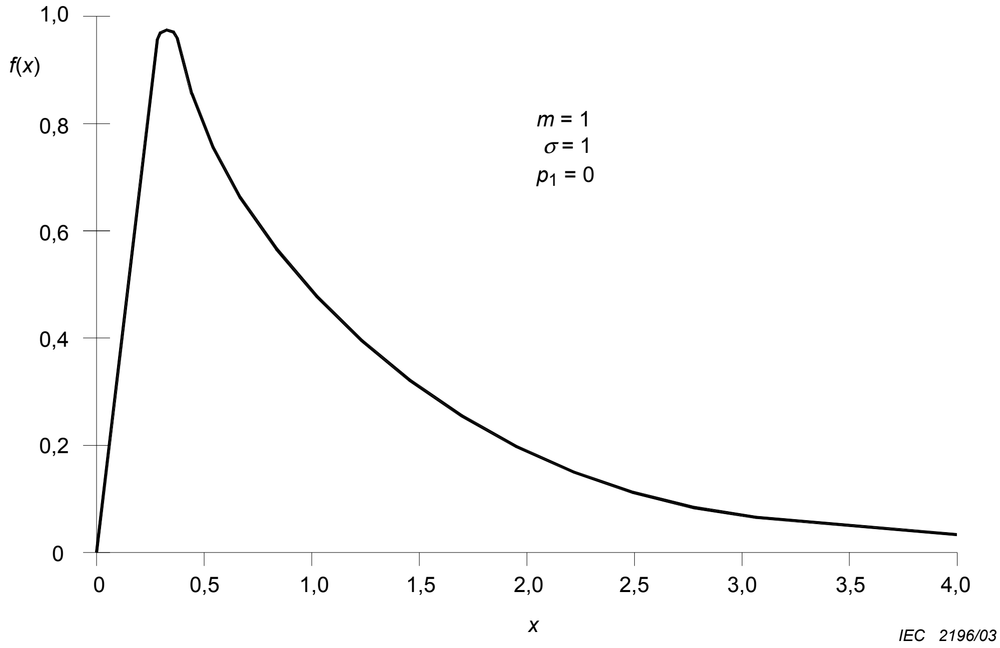


Figure C.2 – Probability density function of standardized log-normal distribution

C.4 Gumbel distribution

C.4.1 General format

Probability density function:

$$f_{(x)} = \frac{1}{p_2} \exp\left[- \frac{x - p_1}{p_2} - \exp\left(- \frac{x - p_1}{p_2} \right) \right], p_2 > 0 \tag{C.19}$$

Cumulative distribution function:

$$F_{(x)} = \exp\left[- \exp\left(- \frac{x - p_1}{p_2} \right) \right] \tag{C.20}$$

C.4.2 Standardized format

Variable change for the standardized format:

$$u = \frac{x - p_1}{p_2} \tag{C.21}$$

Probability density function:

$$f_{(u)} = \exp[- u - \exp(- u)] \tag{C.22}$$

Cumulative distribution function:

$$F_{(u)} = \exp[-\exp(-u)] \tag{C.23}$$

Relation to return period T or exclusion limit e:

$$X(T) = p_1 - p_2 \ln [-\ln (F_T)] \tag{C.24}$$

$$X(e) = p_1 - p_2 \ln [-\ln (F_e)] \tag{C.25}$$

Definition of parameters:

$$p_2 = \frac{\sigma}{C_1}$$

$$p_1 = m - C_2 p_2 = m - \frac{C_2}{C_1}\sigma \tag{C.26}$$

The parameters C_1 and C_2 depend on the number of values considered in a measurement series. They may be calculated as follows.

For a measurement period of n years, z_i values can be calculated as follows:

$$z_i = -\ln(-\ln \frac{i}{n+1}), 1 \leq i \leq n \tag{C.27}$$

$$C_2 = \bar{z} = \frac{1}{n}\sum_{i=1}^{n} z_i \tag{C.28}$$

$$C_1 = \sigma_z = \sqrt{\frac{1}{n}\sum_{i=1}^{n} z_i^2 - \bar{z}^2} \tag{C.29}$$

For simplification, the approximation of an infinite number of observations ($n \Rightarrow \infty$) can be taken, then:

$$C_1 = \frac{\pi}{\sqrt{6}} = 1{,}28255$$
$$C_2 = 0{,}577216 \quad \text{(Euler constant)}$$

Table C.1 gives parameters C_1 and C_2 for a selection of n values if an exact consideration of the number of observations is required.

Table C.1 – Parameters C_1 and C_2 of Gumbel distribution

n	C_1	C_2	C_2/C_1
10	0,94963	0,49521	0,52148
15	1,02057	0,51284	0,50250
20	1,06282	0,52355	0,49260
25	1,09145	0,53086	0,48639
30	1,11237	0,53622	0,48205
35	1,12847	0,54034	0,47882
40	1,14131	0,54362	0,47631
45	1,15184	0,54630	0,47428
50	1,16066	0,54854	0,47261
∞	1,28255	0,57722	0,45005

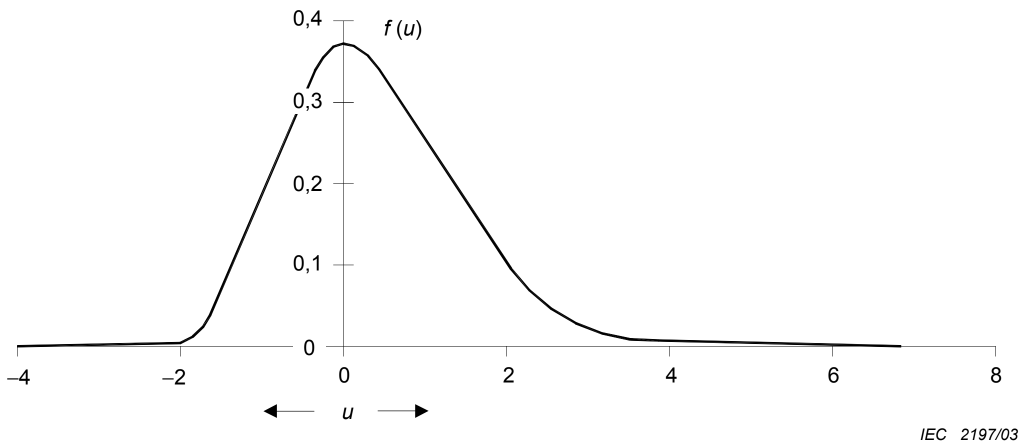


Figure C.3 – Probability density function of standardized Gumbel distribution

C.5 Weibull distribution

C.5.1 General format

Probability density function:

$$f(x) = \frac{p_3}{p_2}\left(\frac{x-p_1}{p_2}\right)^{p_3-1} \times \exp\left[-\left(\frac{x-p_1}{p_2}\right)^{p_3}\right], \quad x > p_1, p_2 > 0, p_3 > 0 \qquad \text{(C.30)}$$

Cumulative distribution function:

$$F_{(x)} = 1 - \exp\left[-\left(\frac{x - p_1}{p_2} \right)^{p_3} \right] \tag{C.31}$$

C.5.2 Standardized format

Variable change for the standardized form:

$$u = \frac{x - p_1}{p_2} \tag{C.32}$$

Probability density function:

$$f_{(u)} = p_3 u^{p_3 - 1} \exp[-up_3], u > 0, p_3 > 0 \tag{C.33}$$

Cumulative distribution function:

$$F_{(u)} = 1 - \exp\left[-u^{p_3}\right] \tag{C.34}$$

Relation to return period T or exclusion limit e:

$$X(T) = p_1 + p_2 \left[-\ln((1 - F_T)\right] \frac{1}{p_3} \tag{C.35}$$

$$X(e) = p_1 + p_2 \left[-\ln(1 - F_e)\right] \frac{1}{p_3} \tag{C.36}$$

Relation to mean value m and standard deviation σ:

$$m = p_1 + p_2 \Gamma\left(1 + \frac{1}{p_3}\right) \tag{C.37}$$

$$\sigma^2 = p_2^2 \left[\Gamma\left(1 + \frac{2}{p_3}\right) - \Gamma^2\left(1 + \frac{1}{p_3}\right) \right] \tag{C.38}$$

where Γ is the Gamma function (see Clause C.8).

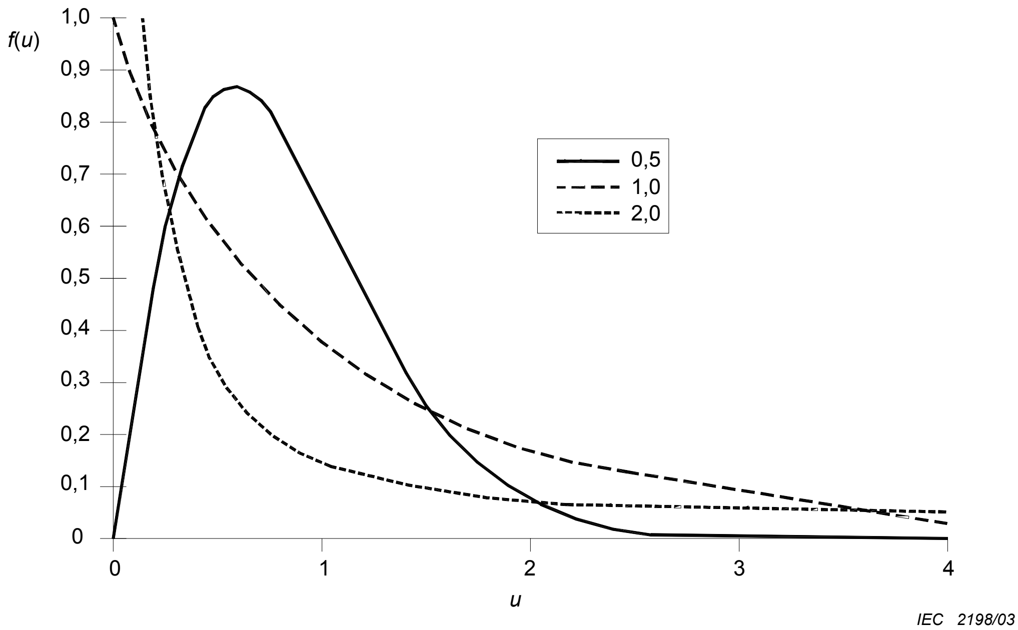


Figure C.4 – Probability density function of standardized Weibull distribution for parameter p_3 = 0,5; 1,0 and 2,0

C.6 Gamma distribution

C.6.1 General format

Probability density function:

$$\mathrm{f}(x) = \frac{1}{p_2 \Gamma(p_3)} \left(\frac{x - p_1}{p_2} \right)^{p_3 - 1} \times \exp\left[- \left(\frac{x - p_1}{p_2} \right) \right], x > p_1, p_2 > 0, p_3 > 0 \tag{C.39}$$

Cumulative distribution function:

$$F(x) = \frac{1}{p_2 \Gamma(p_3)} \int_{p_1}^{x} \left(\frac{x - p_1}{p_2} \right)^{p_3 - 1} \times \exp\left[- \left(\frac{x - p_1}{p_2} \right) \right] dx \tag{C.40}$$

C.6.2 Standardized format

Variable change for standardized form:

$$u = \frac{x - p_1}{p_2} \tag{C.41}$$

Probability density function:

$$f_{(u)} = \frac{u^{p_3 - 1}}{\Gamma(p_3)} \exp(-u), u > 0, p_3 > 0 \tag{C.42}$$

Cumulative distribution function:

$$F_{(u)} = \frac{1}{\Gamma(p_3)} \int_0^u u^{p_3 - 1} \exp(-u) du \qquad (C.43)$$

or

$$F_{(u)} = \frac{\Gamma_u(p_3)}{\Gamma(p_3)} = \mathrm{I}(u, p_3 - 1) \qquad (C.44)$$

Second variable change:

$$u_1 = \frac{u}{\sqrt{p_3}} \qquad (C.45)$$

$$F_{(u_1)} = \Gamma(u_1, p_3 - 1) \qquad (C.46)$$

Relation to return period *T* or exclusion limit *e*:

Since the form of the $\mathrm{I}(u, p)$ function is not analytical, it is preferable to refer to tables to determine the value of u_1. An interpolation of the above mentioned function covering the expected range of uses for the purpose of this standard is given in Table C.2.

$$u(T) = u_1 \sqrt{p_3}$$

$$X(T) = p_1 + p_2 u$$

$$X(T) = p_1 + p_2 u_1(T) \sqrt{p_3}$$

$$X(e) = p_1 + p_2 u_1(e) \sqrt{p_3}$$

Value of parameters: $p_2 = \dfrac{\sigma^2}{m - p_1}$, $p_3 = \left[\dfrac{m - p_1}{\sigma}\right]^2$

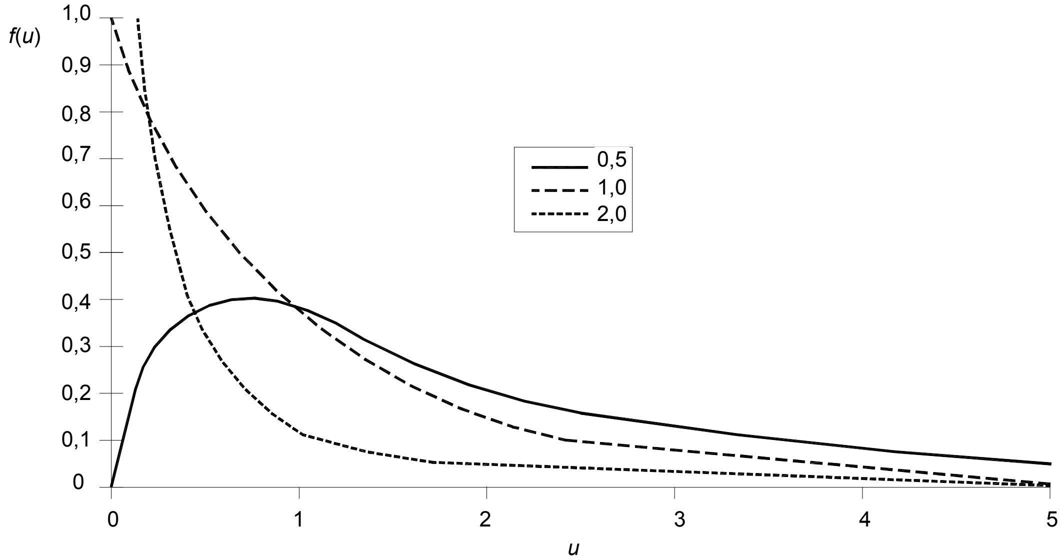


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Figure C.5 – Probability density function of standardized Gamma distribution for parameter p_3 = 0,5; 1,0 and 2,0

Table C.2 – Values of u_1 for given values of function $F_{(u_1)} = \mathrm{I}(u_1,p_3\text{-}1)$

	p_3	0,05	0,10	0,15	0,20	0,25	0,30	0,35	0,40	0,45	0,50
T	F_T										
50	0,98	3,06	3,54	3,68	3,74	3,77	3,79	3,80	3,81	3,82	3,83
150	0,993	6,04	5,95	5,78	5,64	5,53	5,44	5,36	5,30	5,25	5,20
500	0,998	9,86	8,87	8,27	7,86	7,55	7,32	7,14	6,99	6,86	6,75

	p_3	0,55	0,60	0,65	0,70	0,75	0,80	0,85	0,90	0,95	1,00
T	F_T										
50	0,98	3,84	3,84	3,85	3,86	3,87	3,88	3,88	3,89	3,90	3,91
150	0,993	5,17	5,14	5,11	5,09	5,07	5,05	5,04	5,03	5,02	5,01
500	0,998	6,66	6,58	6,52	6,46	6,40	6,36	6,31	6,28	6,25	6,22

	p_3	1	1,1	1,2	1,3	1,4	1,5	1,6	1,7	1,8	1,9	2
e	F_e											
2 %	0,02	0,021	0,026	0,033	0,042	0,053	0,067	0,084	0,104	0,116	0,13	0,145
5 %	0,05	0,053	0,066	0,083	0,104	0,122	0,141	0,161	0,184	0,207	0,227	0,248
10 %	0,10	0,106	0,131	0,157	0,183	0,211	0,238	0,265	0,293	0,32	0,347	0,375

	p_3	2,1	2,2	2,3	2,4	2,5	2,6	2,7	2,8	2,9	3
e	F_e										
2 %	0,02	0,162	0,181	0,202	0,217	0,233	0,25	0,268	0,289	0,308	0,324
5 %	0,05	0,271	0,295	0,316	0,337	0,359	0,382	0,406	0,426	0,447	0,469
10 %	0,10	0,403	0,429	0,455	0,482	0,509	0,534	0,559	0,585	0,611	0,635

C.7 Beta distribution, first type

C.7.1 General format

Probability density function:

$$f(x) = \frac{\Gamma(p_2 + p_3)}{\Gamma(p_2)\Gamma(p_3)}(x - p_1)^{p_3 - 1}[1 - (x - p_1)]^{p_2 - 1}, x > p_1, 0 \le p_1 \le 1, p_2 > 0, p_3 > 0 \qquad \text{(C.47)}$$

$\Gamma(x)$ is the Gamma function (see Clause C.8, Equation (C.58))

C.7.2 Standardized format

Variable change for the standardized form:

$$u = x - p_1 \qquad \text{(C.48)}$$

Probability density function:

$$f(u) = \frac{\Gamma(p_2 + p_3)}{\Gamma(p_2)\Gamma(p_3)} u^{p_3 - 1}[1 - u]^{p_2 - 1}, 0 \le u \le 1, p_2 > 0, p_3 > 0 \qquad \text{(C.49)}$$

$$m = p_1 + \frac{p_3}{p_2 + p_3}(1 - p_1) \qquad \text{(C.50)}$$

$$\sigma^2 = (1 - p_1)^2 \frac{p_2 p_3}{(p_2 + p_3)^2 (p_2 + p_3 + 1)} \qquad \text{(C.51)}$$

C.7.3 Applications

The Beta distribution is characterized by having a lower and an upper limit. The parameter p_1 in Equation (C.47) represents the lower limit, while the upper limit is set to 1.

In some cases it may be advantageous to use the following transformations:

$$p_1 = a;\ p_3 = r;\ p_2 = t - r,\ x = U;$$

then from Equation (C.47) follows

$$f(U) = \frac{\Gamma(t)}{\Gamma(r)\Gamma(t - r)}(U - a)^{r - 1}(1 - (U - a))^{t - r - 1} \qquad \text{(C.52)}$$

The variable U has a lower limit a, different from 0 with $0 < a < 1$.

The mean value $\overline{U}$ and standard deviation σ_U of U can be obtained from:

$$\overline{U} = a + \frac{r}{t}(1 - a) \qquad \text{(C.53)}$$

$$\sigma_U^2 = (1-a)^2\, r(r-t) \;/\; t^2(t+1) \tag{C.54}$$

For a = 0 it is obtained:

$$f(U) = \frac{\Gamma(t)}{\Gamma(r)\Gamma(t-r)} U^{r-1}(1-U)^{t-r-1} \tag{C.55}$$

where $0 \le U \le 1$

In addition, the mean value $\overline{U}$ and the standard deviation σ_U of variable U can be calculated from Equations (C.53) and (C.54):

$$\overline{U} = \frac{r}{t} \tag{C.56}$$

$$\sigma_u^2 = \frac{\overline{U}\left(1-\overline{U}\right)}{t+1} = \frac{r(t-r)}{t^2(t+1)} \tag{C.57}$$

Figure C.6 shows examples for a = 0; r = 5,0; t = 5,5; t = 6,0; t = 7,0

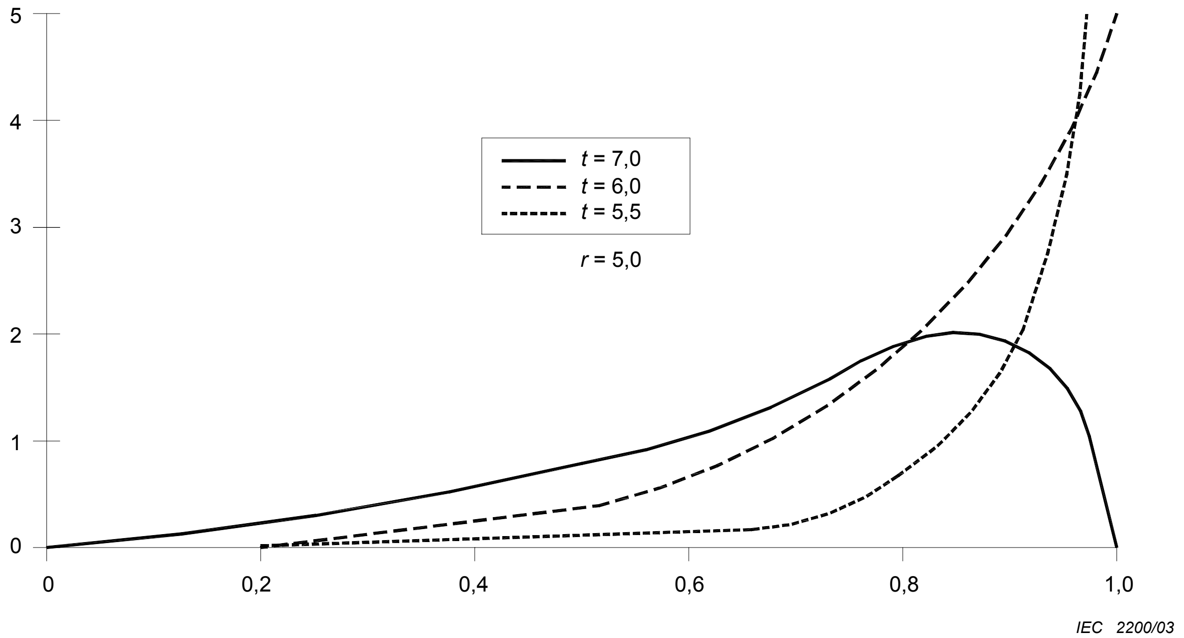

Figure C.6 – Probability density function of standardized beta distribution for parameters *r* = 5,0, *t* = 5,5; 6,0 and 7,0

C.8 Gamma function and its relationships

The Gamma function is used in the expression of various distributions described above.

Its main features are recalled hereafter.

Definition:

$$\Gamma(p) = \int_0^\infty e^{-x} x^{p-1} dx \tag{C.58}$$

Relationships:

$\Gamma(p + 1) = p\Gamma(p)$

$\Gamma(n + 1) = n_1, n \in \aleph$

$\Gamma(1) = 1$

$\Gamma(\frac{1}{2}) = \sqrt{\pi}$

An approximation of its numerical values is given by the 1st order Stirling formula:

$$\Gamma(p) = e^{-p} p^{p-\frac{1}{2}} \sqrt{2\pi}(1 + \frac{1}{12p}), p > 1 \tag{C.59}$$

In the cases where $0<p<1$ $\Gamma(p)$ can be obtained from:

$$\Gamma(p) = \frac{\Gamma(p+1)}{p} \tag{C.60}$$

or, for a better approximation:

$$\Gamma(p+2) = (p+1)\Gamma(p+1) \Rightarrow \Gamma(p) = \frac{\Gamma(p+2)}{p(p+1)} \tag{C.61}$$

The incomplete Gamma function and the related I function are used in the expressions of the Gamma distribution.

Definition of incomplete function Γ_x:

$$\Gamma_x(p) = \int_0^x e^{-x} x^{p-1} dx \tag{C.62}$$

Definition of I function:

$$\mathrm{I}(x,p) = \frac{\Gamma_x(p+1)}{\Gamma(p+1)} \tag{C.63}$$

Variable change:

$$u = \frac{x}{\sqrt{p+1}} \tag{C.64}$$

Relationship:

$$\mathrm{I}(u,p) = \mathrm{I}(x,p) \tag{C.65}$$

Table C.2 gives the values of u as a function of I and $p_3 = p + 1$.

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